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By G. H. DUGGAN, M.Can.Soc.C.E., Past President

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Please read and send in as full discussion as possible at earliest date

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A paper describing the superstructure of the Quebec Bridge is in course of preparation by the Engineers of the St. Lawrence Bridge Company. It will however be some time before the paper is presented as many calculations must be abbreviated, and the working drawings, and many of the drawings for the construction equipment, as well as those issued for instruction to the shop and the erection force, must be condensed into forms that will come within the compass of an ordinary paper.

The paper is being written in order that the members of the Society may have for reference a technical description of the structure as built, and of the special plant and methods employed in its manufacture and erection, but it will not come within the scope of the paper to discuss the designs prepared for tender or the considerations that led to the adoption of the final design.

There are however some engineering considerations in connection with the preliminary designs and trial work leading up to the final design which, while beyond the scope of the paper, may be interesting to the members of the Society. Moreover, I feel that the Society should have some record of those who actively contributed to the success of the undertaking because so many engineers have been employed upon the work, and it has been so unusual, and of such long duration, that a mere statement of those in charge of various Departments at the finish of the work would omit the names and responsibilities of some who should be recorded in the history of the undertaking.

As it is probable that I have the most intimate knowledge of all phases of the work from the time our designs for tenders were started until its completion, the duty of giving this information and record seems to devolve upon me, and no better way of presenting it occurs to me than to tell the story of our work, with a discussion of our designs up to the point where the description will be taken up by the forth-coming paper.

A brief history of the preliminaries, and the award of the contract to the St. Lawrence Bridge Company, will assist in an understanding of the competition and the designs.

The project of bridging the St. Lawrence at Quebec is an old one, but its early history has no bearing on the present Quebec Bridge, and may be neglected. Tenders were called by the Quebec Bridge & Railway Company in 1899, which later resulted in the award of the contract to the Phoenix Bridge Company for the bridge which failed on August 29th, 1907.

Immediately after the accident the Dominion Government appointed a Royal Commission to report upon it, and after receiving the report, appointed a Board of Engineers to prepare a new design for the bridge. I was told by the then Minister of Railways and Canals that in appointing this Board it had been the desire of the Government to select Engineers best qualified by their experience in long span bridges to deal with this unusual problem, and to appoint one engineer from Canada, one from Great Britain, and one from the United States. After advisement he had appointed Mr. H. E. Vautelet, M. Can. Soc. C. E., for a long time Bridge Engineer of the Canadian Pacific Railway, as Chairman and Chief Engineer; Mr. (now Sir Maurice) FitzMaurice, C.M.G., M.I.C.E., of London, England, who had been one of the engineers on the staff of the Forth Bridge; and from the United States, Mr. Ralph Modjeski, M. Am. Soc. C.E., who had been connected with many long span bridges.

The report of the Royal Commission appointed to investigate the failure of the Phoenix Bridge in 1907 is very comprehensive, and goes beyond the mere taking of evidence and the investigation of the faults of the bridge, as the Commission assembled most of the available data on other long span bridges, illustrated their important features, recorded the tests on large size compression members that had any bearing upon the work, and made a number of tests to supply some lacking experimental data of the behavior of large compression members under stress.

The Board of Engineers continued the investigations of the Royal Commission and made a number of trial designs. Mr. Vautelet selected one of these trial designs and brought it to the condition of a working design about the end of 1909. It was his intention to make this the Official Design on which the bridge was to be built, and to call tenders for the construction on it only. The other members of the Board did not consider the design

to be in all respects a desirable one and consented to tenders being called upon it only on the condition that contractors would be allowed to submit tenders on their own designs if they so desired and, probably due to these disagreements, tenders were not actually advertised until the 17th June 1910. Before tenders were called and during the discussion on the design, Mr. FitzMaurice resigned from the Board. Mr. Chas. MacDonald, M.Can.Soc.C.E., a noted bridge builder, a Canadian by birth and permanent residence, but then retired from active practice and spending much of his time in the United States, consented to become a member of the Board until the Contract could be awarded.

After tenders were received, Mr. Vautelet still strongly contended for his design while his colleagues, Mr. Modjeski and Mr. MacDonald, favored the design of the St. Lawrence Bridge Company. To settle this dispute the Minister, as provided for in the original Order-in-Council, called in, with Mr. Vautelet's consent, two additional Engineers, Messrs. M. J. Butler, C.M.G., Past President Can.Soc.C.E. and Henry Hodge, Am.Soc.C.E. of New York, to assist the Board in coming to a decision. Four engineers of the Advisory Board recommended the design of the St. Lawrence Bridge Company, M. Vautelet only dissenting. Mr. Vautelet resigned when his colleagues' recommendation was accepted about the end of February 1911.

Messrs. Butler and Hodge were appointed to the Board only to assist in deciding upon a design and specifications and their duties ceased with the signing of the Contract. Mr. MacDonald had also made it a condition that he should be relieved when the design was arranged and the contract awarded, and after the Contract was signed, on April 4th, 1911, Mr. Modjeski was the only member left of the Board. About a month later, Mr. C. N. Monsarrat, M.Can.Soc.C.E., was appointed to the position made vacant by Mr. Vautelet's resignation, and shortly after Mr. C. C. Schneider, Past President of the Am.Soc.C.E., joined the Board. Mr. Schneider was regarded as the Dean of Bridge Engineers in America and was a valued member of the Board until his death in 1916. When Mr. Schneider died, Mr. H. P. Borden, M.Can.Soc.C.E., who had been Secretary of the Board, was appointed to fill the vacancy.

Sometime before this Mr. Phelps Johnson, President of the Dominion Bridge Company, Past President of the Society, had arranged with Mr. F. C. McMath, M.Can.Soc.C.E., President of the Canadian Bridge Company, that in view of the magnitude of the work, and the interest of Canadians in making the work a Canadian enterprise, the Dominion and Canadian Bridge Companies should combine their forces in the organization of a special Company to tender on the bridge, and, if successful, to carry out its construction, each Company taking a half interest in the venture and contributing such of its staff as might be necessary to make

an organization for the new Company. This Company was later incorporated as the St. Lawrence Bridge Company. Prior to this Mr. G. F. Porter, M. Can. Soc. C.E., Chief Draftsman of the Canadian Bridge Company, and Mr. P. L. Pratley, M. Can. Soc. C.E., of the Engineering Staff of the Dominion Bridge Company, had been released to the Board of Engineers to assist in the preparation of the official design.

The magnitude of the disaster to the bridge being erected by the Phoenix Bridge Company, with its lamentable loss of life and serious financial loss, coupled with the fact that the bridge was larger than anything that had heretofore been attempted and the probable very heavy cost of constructing the bridge in a proper manner, had caused serious misgivings in the minds of the public and the Government as to the practicability of the undertaking, and from the outset the Government had safeguarded itself in every possible way.

A prominent clause of the contract read as follows:

"The Contractor must satisfy himself as to the sufficiency and "suitability of the design, plans and specifications upon which the bridge "is to be built, as the Contractor will be required to guarantee the satisfactory "erection and completion of the bridge, and it is to be expressly understood "that he undertakes the entire responsibility not only for the materials "and construction of the bridge, but also for the design, calculations, "plans and specifications, and for the sufficiency of the bridge for the "loads therein specified. And the enforcement of any part, or all parts, "of the specifications shall not in any way relieve the Contractor from "such responsibility".

To implement the above guarantee the St. Lawrence Bridge Company was obliged to make a cash deposit of $1,297,500., and in addition both the Canadian and Dominion Bridge Companies signed guarantees for the completion of the bridge putting their whole assets at stake.

The discussion of the designs will be addressed to those who have not made special study of long span bridges, and it is hoped those who are conversant with the subject will forgive if taken over familiar ground, or if the subject seems to be treated in too elementary a manner.

PLATE I

shows in outline the elevation of—

(1) The design of the Phoenix Bridge Company which failed in 1907;

(2) The Official Design when tenders were advertised, but afterwards known as Design 1, and supplemented by Designs up to V before tenders closed.

PLATE II

shows in outline the elevation of the great cantilever bridges that had been built up to that time.

It will be seen that all existing bridges, except the Forth, were of so much less span than the Quebec Bridge that the chief guides or precedents for the problems in hand were the Forth Bridge, and the Phoenix Bridge which had failed. Many good features of the Forth Bridge were not applicable for reasons that will be given later.

The report of the Royal Commission which investigated the failure of the Phoenix Bridge, brought out clearly many faults of that design that could easily be corrected in a new design, some of these being the small width centre to centre of truss, the very high unit stresses, the curved compression chords, the inadequate lacing of compression members and the poor splice connections of the members.

The report also disclosed the use of open joints during erection. In our view the failure of the Phoenix Bridge may be ascribed chiefly to these open joints, and as open joints are difficult to avoid in designs requiring sub-divided main panels, it may be well to discuss this subject briefly.

PLATE III

taken from the report of the Royal Commission, shows the deformatiom diagram of the anchor arm of the Phoenix truss, and the open joints in erection.

In all framed structures the lengths of the members as manufactured or before entering into the structure, differ from the lengths they will have in the completed structure, due to the elongation or compression, as the case may be, induced by the stresses to which they are subjected. It is customary to make provision for this in the "framed lengths" of the members, by calculating the amount of the extension or compression, and diminishing or increasing the length of the member so that it may have its goemetrical length in the structure after the stresses are imposed. Thus, if the members were assembled on their sides on a flat surface without any stresses in them, the whole truss would be considerably distorted from the form it would have when erected, as shown by the heavy lines of the drawing.

In cantilever erection the members are necessarily placed without load upon them and until the work has proceeded a considerable distance the truss will approximate to the form which it would a. pt if laid down flat. As the load comes on, the truss is gradually brought toward the form it will finally have, when the weight of the centre span is attached to the end of the cantilever.

In certain forms of trusses with sub-divided panels, the "framed lengths" which will bring the truss to its proper form when under full load, cause some of the members to take a considerable bend during erection although they will eventually be straight when under load. As erection requirements cause these members to be put up in sections and then spliced at the joints, they cannot be assembled easily without allowing all the bend to come where they are spliced, thus causing a wedge shaped opening at the joint with the sections of the member only bearing at one edge. As the load comes on and the structure deflects the members straighten and the joints gradually close, but before the joint comes to a true bearing over its whole surface, the intensity of the pressure on one edge is very great, and may cause failure, as in the case of the Phœnix Bridge.

Diag. 2 of Plate I shows the original design prepared by Mr. Vautelet

PLATE IV

copied from the Official Drawing, shows the anchor arm to a larger scale.

PLATE V

illustrates the cantilever arm and suspended span to the same scale.

The truss had a main web system of the single warren or triangular type, each truss panel being sub-divided to give a point of support for an intermediate floor beam, and thus make the floor system panels of moderate length. This sub-division was accomplished by means of a triangular frame suspended from the mid-heights of the main diagonals. This system of bracing overcame in large measure the local deformations and secondary stresses of the ordinary sub-divided panels, but it required an adjustment in the top chorp of the triangular frame, as the framed length of this chord during erection would be different from its final length when the whole truss was completed and had taken its normal deflection.

Figs. 1-1a-1b-1c show in outline the progressive erection of the cantilever arm of the Official Design.

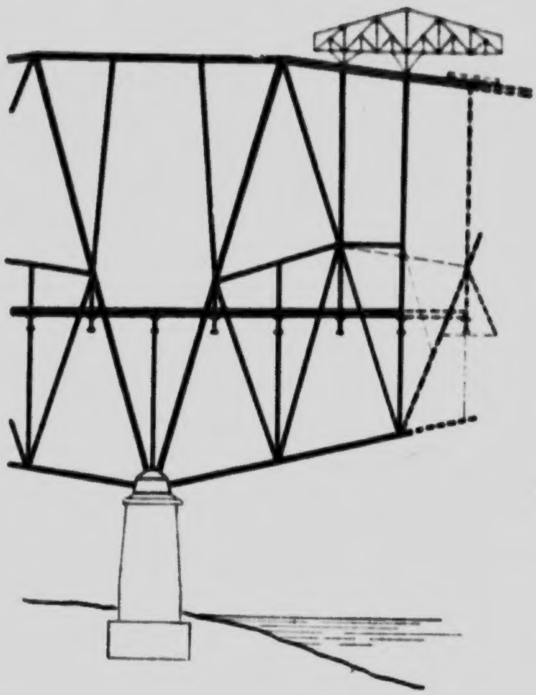

Fig. 1

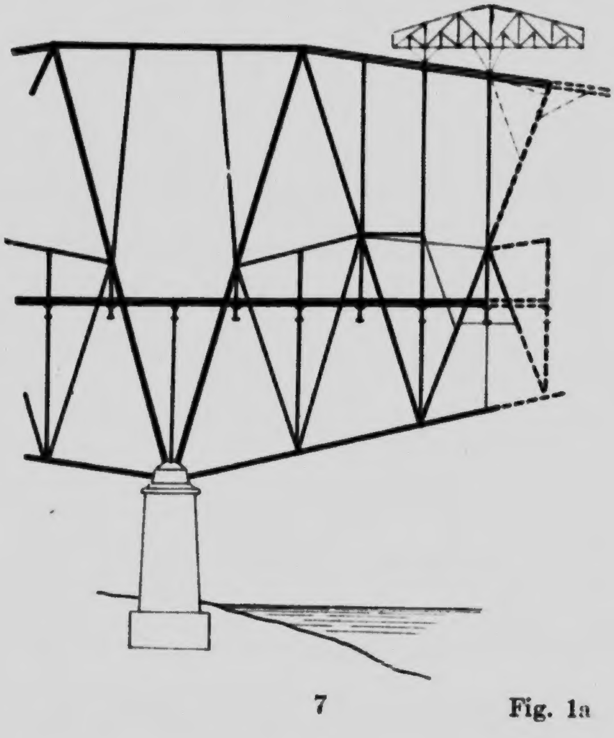

Fig. 1a

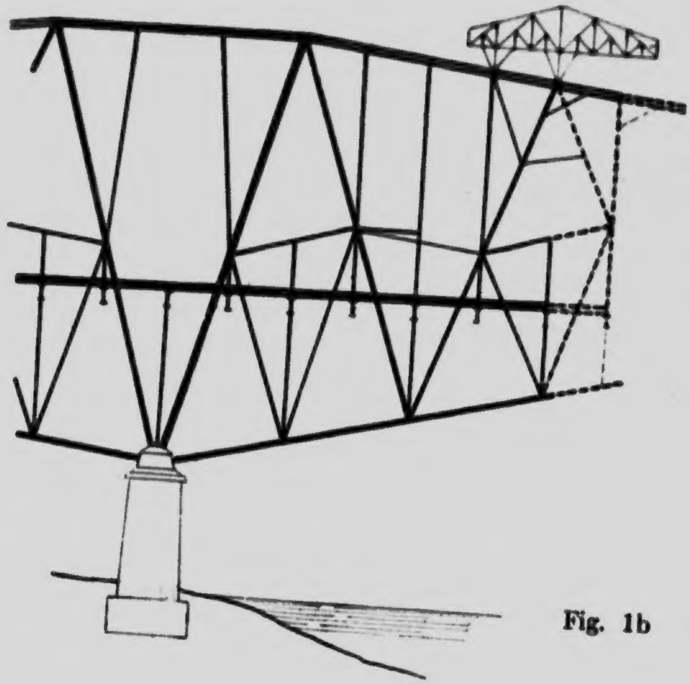

Fig. 1b

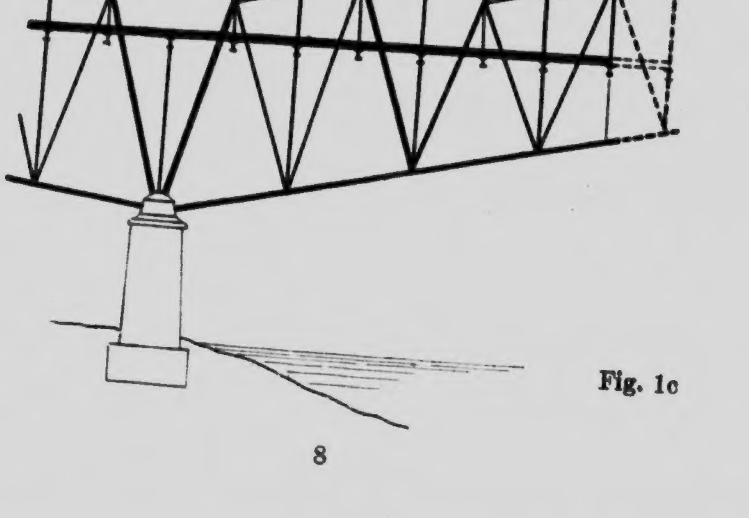

Fig. 1c

Mr. Vautelet had worked out two schemes of erection for this bridge, the two schemes differing only in the method of erecting the anchor arm.

In the first it was intended to erect the anchor arm complete on false-work; while in the second it was intended to place a temporary supporting pier at the first main panel point shorewards from the main pier, erect this panel on false-work and then erect both the River and anchor arms as cantilevers—each arm to be carried out at such a rate that they would always balance over the base formed by the main and temporary piers.

In either case, the scheme of cantilever erection was to use a top chord traveller starting at the main pier and working out panel by panel.

The general form of the truss prevented any panel from being self-supporting until a main panel point had been reached, and it was intended to overcome this, by making use of the compression chord of the triangular sub-truss frame as the upper tension member of a cantilever that would hold up the lower half of the second main panel until the long tension diagonal could be connected, this small cantilever being anchored by means of tie-bars to the corresponding point of a completed panel in the anchor arm. Similarly, when the third panel was being erected, temporary tie-bars were to be placed between the chords of the first and second sub-trusses; in the first instance to hold the long compression diagonal during erection, and afterwards to make use of the sub-truss as a cantilever as in the first panel. The principle was to be continued throughout the cantilever arm.

This method of erection required that the upper chords of the sub-frames should be designed to carry their compression stresses when the bridge was completed, and should in addition be capable of carrying the heavy tension stresses imposed during erection. Moreover, and herein lay a considerable difficulty, they should be capable of being adjusted with nicety to varying lengths while under heavy stresses.

A consideration of the Official Design in the light of the investigations and the data assembled by the Royal Commission and by the Board of Engineers assured us that Mr. Vautelet had put on paper a bridge in which, if built, every confidence could be placed that it would perform the work for which it was designed. It was, however, manifest that many of the members would be much too large to manufacture with any existing equipment, and that the manufacture must be carried out with a degree of accuracy hitherto unattained to assure that the parts would go together in the structure and perform their intended functions properly. This requirement could, however, be fulfilled as it only needed proper equipment and exceptionally good workmanship, the serious difficulty being the erection or the assembling of the members to construct the Bridge.

When I returned to my old position as Chief Engineer of the Dominion Bridge Company in January 1910, all the essential details of the official design had been worked up, inquiries were being made as to the largest

sizes of material obtainable and possible methods of erection were being considered. Mr. Johnson had followed the work of the Board, and when it was decided that the Dominion and Canadian Bridge Companies would tender jointly for the construction of the Bridge, he had given much thought to the problems of its construction. He had early foreseen such great difficulties in manufacture and erection that he considered desirable some radical departures from Mr. Vautelet's design or, indeed, from any design for a long span bridge that had heretofore been illustrated. Mr. Johnson's studies led him to an appreciation of the possibilities of what has since come to be known as the "K" form of bracing, never before used for an important structure and as he and his staff were at the time fully occupied with other important work, he asked me to take up the development of this design and generally supervise the preparation of tenders.

Mr. F. P. Shearwood, M. Can. Soc. C.E., Assistant Chief Engineer of the Dominion Bridge Company, had been collaborating with Mr. Johnson in considering the erection of the Official Design and the possibilities of the alternative "K" design. He had already made a number of preliminary sketches and outline strain sheets, and it was unfortunate that Mr. Shearwood's regular duties prevented him from following up this work. He was, however, available for consultation and was of much assistance throughout the development of the "K" design.

The designing staff was built up as rapidly as possible, Messrs. Harkness, Wilson and Linton of the Dominion Bridge Company Staff early being transferred to it. As the work developed the staff was added to until when Mr. Porter and Mr. Pratley returned from the Board of Engineers, about May 1910, the staff had outgrown the available accommodation and a new office was built in which the preparation of our designs could be carried out. The Staff consisted of Messrs. G. F. Porter, M. Can. Soc. C.E.; P. L. Pratley, M. Can. Soc. C.E., A. L. Harkness, A.M. Can. Soc. C.E., L. R. Wilson, A. M. Can. Soc. C.E. (now Major); E. C. Kerrigan, A.M., Can. Soc. C.E., H.M. Lamb, A.M. Can. Soc. C.E. (now Professor): A. P. Linton, A.M. Can. Soc. C.E., (now Major): Jas. McNiven, A.M. Can. Soc. C.E. (also overseas) and F. C. MacDonald, with frequent assistance of others from the Drawing Office in the making of tracings and the computation of weights and ordinary stresses.

The work was so unusual, with so little precedent to guide, that it required a large staff of engineers to carry it on.

In preparing competing designs for ordinary bridges there is so much data at hand as to the cost of shop manufacture, erection costs, and the weight of details, that it is generally unnecessary to do more than prepare strain sheets with a few details from which weights can be very closely estimated; but, the magnitude of this bridge required, in addition to calculations for dead and live load stresses, extended calculations of wind, temperature and traction stresses and also of the bending stresses due to the weight of the unsupported length of the members, and to the

elastic deformation of the structure or what are usually termed secondary stresses. The make-up of the members, as well as the details for connecting the several members together, also necessarily differed much from previous practice with smaller members.

The above considerations made it impracticable to follow the ordinary course of balancing the merits of trial designs by comparing outline strain sheets, and for every design which was considered worth a real trial it was necessary, after fixing the length of the suspended span and deciding how it might be erected, to work back from the ends of the cantilever arms to the piers, designing and detailing each member with sufficient accuracy to obtain a very close approximation of the final weight of the member in the structure, and particularly to consider how the larger members could be manufactured and erected in the bridge.

Concurrently with the design of the bridge itself it was therefore necessary to keep in view the design and cost of the plant and equipment for manufacturing the work, the transportation of the members to the site and the design and cost of the erection plant necessary for the design in hand. Included in the latter are the special erection travellers, steel erection staging, pontoons and storage yards with heavy cranes, in addition to the ordinary erection equipment required for a large bridge.

Our preparations are evidence that we were most anxious to obtain the contract, but we were equally anxious, if successful in our tender, that the Bridge should have a pleasing apperance, that none of the errors of the Phoenix design should be repeated, that the make-up of the members, the details and the connections should be of the most efficient character and, indeed, that we might be confident the bridge would be in all respects a credit to us and to Canadian bridge building.

A superficial examination of the Official Design had revealed that it was the result of much careful work, and that it had many excellent features. It was therefore felt that before making, or concurrently with the preparation of, our alternative designs, a careful detailed study of Mr. Vautelet's design would be of much assistance in developing our own work. The critical examination confirmed the opinion that in many respects, little or no improvement could be made. The principal features that were adopted, to be incorporated in the alternative designs were:

(a) The general form of the compression members with abutting joints fully spliced before being subjected to stress;

(b) The compression chords without bends in their length but increasing in depth towards the piers to keep a proper ratio of thickness of material as the stresses increased.

The curved bottom chords of the Phoenix design while perhaps tending to economy of material, presented several very objectionable features, the principal being that the horizontal wind forces, and these are very considerable, cause heavy vertical components at the joints, reversing in direction as the chord is in tension or compression. These do not exist with the chords lying in one plane from end to end. It is most difficult if not impracticable to fully or efficiently splice chords deflected at panel points.

(c) All of the main compression members in the Official Design were made up of four webs, shop connected in pairs, so that each truss was virtually two trusses placed close together and connected by tie-plates and lattice bars in the field. This assured a better distribution of stress throughout the members of the truss, permitted the heavy members to be shipped in practical lengths and greatly facilitated the erection.

(d) The construction of the shoe and transfer of the load from the steel work to the masonry. The total vertical load was estimated at 30,500 tons; the transverse wind load at 790 tons, and the longitudinal wind load at 3,730 tons; a horizontal compression load of 15,200 tons. The loads required a bearing area on the masonry of 700 square feet, and there were many difficulties in the way of evenly distributing such large forces over so great an area while at the same time providing for the transverse forces. The pins were so placed that each loaded the area of the bed-plate tributary to it and the design amply provided for all the forces as well as the necessary stability.

(e) The sleeves on the pins to reduce the friction and thus provide for the necessary deflection during erection were also good.

The displacement diagrams were good as well as the secondary stresses resulting from deformation.

The objectionable features from the theoretical view point and from that of construction will be referred to in comparing the Official Design with the present Bridge, where it will be shown that we were able to depart from the make up of some members, and from the details and the connections between web members and chords in a manner much to our advantage in shop and erection, and we think also to the betterment of the final structure.

PLATE VI

gives a comparision of the larger chord sections of the present Quebec, Forth and other great cantilever bridges. The section of the Quebec chords is much larger than that of any of the other bridges, having 1940 square inches against the next largest of 853 square inches, and the section of the Official Design was still larger, having 2,038 square inches. The Hell Gate Arch at New York, since constructed, has a large section, 1,392 inches, but that is different type of construction and is not really comparable for our purpose. Similarly, the circular section of the Forth Bridge is not comparable for reasons to be given.

PLATE VII

shows the cross sections of all the important members in the cantilever arm of the bridge as constructed. The bottom chords between panel points were about 86 feet long, and the heaviest section of bottom chord weighed, with its details, about 380 tons. It would be impracticable

to place so large a member with its center of gravity some 45 feet from its connection to the work already built. By splitting it down the longitudinal medial line and splicing it about the middle of its length, each panel of lower chord could be manufactured, shipped and erected in four pieces, none exceeding 93 tons in weight and of which the centre of gravity was only some 25 feet away from the point of its connection or where the erection traveller could stand. The large compression diagonals, while not so great in sectional area, were considerably longer and were really more difficult to handle. The load of these heavy members and the reach necessary to place them, thus became the measure of capacity of the erection traveller.

PLATE VIII

shows the general character of the erection operations of the Official Design, the temporary members for holding up the permanent members of the bridge already placed but not finally connected, and the members that the traveller will place when in the position shown. Although the amount of permanent bridge to be supported by temporary members whould necessarily vary for different positions of the traveller, the principle remains the same throughout the whole erection of the cantilever arm.

PLATE IX

shows the top chord traveller designed for the above method of erection.

Travellers having a reach of one panel, one and one-half panels, and two panels, were each designed and compared, the one illustrated with a two panel reach being adopted as having the minimum of objectionable features.

Work of this magnitude cannot be handled without risk in spite of every precaution, and much consideration was given to adopting methods and plant that promised a maximum of safety. In a general way the short reach traveller is much lighter and the traveller itself is thus safer to handle, but it requires more temporary material for holding up portions of the structure that cannot be permanently connected up, with the traveller in the position for placing these members. It was therefore considered that while the long traveller was more difficult to handle, its use involved less risk than having so much of the permanent structure hanging on temporary adjustable members as would be required with the short traveller. The long traveller also permitted much more rapid work.

It was intended to carry the traveller upon temporary stringers set by the traveller itself on the top of the vertical posts as these were erected, those over which the traveller had passed being picked up and shifted to the forward position as required.

The traveller alone was estimated to weigh about 1,000,000 pounds, with a moving load on its front wheels of nearly 900,000 pounds, and when lifting its maximum loads the reaction of the front supports would be over 1,500,000 lbs. These heavy loads necessitated heavy stringers to carry them, and very heavy stools to rest on the top of the posts on which the traveller could be blocked when at rest and working. They also required considerable increase in the sections of the vertical posts of the truss and in the sway bracing connecting these posts, the normal functions of these posts in the completed structure being merely to carry the weight of the top chord.

It was difficult to plan how the stringers already passed and left to the rear could be picked up, transferred to the front end of the traveller, and set in the position required to supply a track on which the traveller could proceed. There was a decided risk in passing this very heavy traveller down the sloping chord and stopping it at exactly the right point, as any positive stops used could only be of a temporary nature, it being necessary to remove them to allow the traveller to pass on after performing its work at the point in question. It was difficult to give lateral stability to the stringers carrying the traveller with its great reach and large wind surface.

In addition to the above, the operation of the traveller when erecting the members of the bridge presented many serious problems. It has been stated that for the sake of reducing the weight of the members to pieces that could be handled, every main member of each truss was field connected down a medial plane. The general width of the important compression members was about 10 feet out to out, and the width centre to centre about 5′6″. The heavy members to be erected could only be brought out on the floor of the bridge as far as the forward point of support of the traveller. They must then be picked up by the tackles, eased forward and transversely before being hoisted into their exact position. To do this conveniently the tackles should be hung over the centre line of the piece being hoisted, but, if over the centre line of the outside half, the tackles would be 5′6″ too far out from the inside half. Generally this placing is performed by a guy tackle, but with such heavy weights and short drift it would add appreciably to the stresses, and consequent necessary capacity of the hoisting equipment.

The above difficulties were not due to the form of truss or to the general design of the structure beyond its magnitude, the problems arising from the large dimensions of the members both in length and width and the very heavy weights to be handled at a long distance from the point of support. There were, however, other difficulties due to the form of truss; the principal being the large quantity of temporary adjustable members and their connection to the permanent members, together with the exact adjustment under very heavy stress necessary in order to make connections of main panel points. Some of these members would have a stress of 3,300,000 lbs., a heavy force under which to make delicate adjustments.

Other difficulties presented themselves in carrying out the erection step by step and, while none of them were insurmountable, the sum was so great that it seemed worth making every effort to avoid them. I think it was the knowledge that these problems would arise long before the erection was worked out in detail that led Mr. Johnson to the "K" form of bracing.

The Official Design I. was estimated to weigh 72,700 tons, of which 52,000 tons was nickel steel. In very long span bridges a limiting length exists where the structure can only carry its own weight, and is unable to carry any superimposed load. The length naturally depends on the relation existing between the strength of the material under stress and its weight. For ordinary carbon steel construction and the loading specified, the length of the Quebec Bridge span begins to approach this limit. It was therefore of great importance that there should be no unnecessary dead-weight, particularly in the suspended span and at the ends of the cantilevers, where every pound of load, either dead or live, requires several pounds of material in the cantilevers to carry its effect to the supporting piers.

Nickel steel, which was known to be about 40% stronger in tension and was expected to be 40% stronger in compression, although not proved by experiment at that time, was hence a necessity in portions of the structure. It cost, however, at the time the estimates were being made, nearly two and one-half times as much as carbon steel, and it was apparent that if carbon steel were used where it would not affect the sections of the members in the long channel span,—namely: in those members supported directly by the piers and throughout the anchor arm,—while it would considerably increase the total weight of the structure, much economy in cost would result. It was also apparent that there was much unnecessary weight in the long vertical redundant members inserted only to support the weight of the top chord and the erection traveller, but which could not carry any live load nor serve any other useful purpose in the completed bridge. There was promise of economy and improvement in appearance in reducing the depth at the ends of the cantilevers of both the river and anchor arms, and in reducing the lengths of the anchor arms.

PLATE X

taken from Mr. Modjeski's paper on long span bridges, shows a graphic comparison of the weights of the Quebec and Forth Bridges. While this does not enter into the present discussion it shows the very great difference in weight per foot of the centre portions of the bridge, and the portions near the main supporting piers and serves to illustrate the importance of reducing the weight towards the centre of the channel span.

Tenders were advertised on the 17th June 1910, to be submitted on the 1st of September, this date later being extended to the 1st of October 1910. The time allowed would have been altogether too short to prepare alternative designs, but the Board had issued a preliminary specification

at the end of 1909 and the designs and information of the Board were available to contractors who expected to tender, so that in reality there was about eight months in which to prepare tenders. As already stated, the Board had evolved satisfactory forms of compression members and in many instances details that could be used and the Official Design supplied data from which close approximations of weight could be made for alternative designs—indeed the work of the Board was so thorough that practically all existing knowledge on the subject was reviewed and put into workable form.

While Mr. Vautelet and I did not agree on the design of the Bridge, I wish to acknowledge the large contribution made by him to the knowledge of the subject and his readiness to explain his designs and the results of his studies. A history of the undertaking would be incomplete if Mr. Vautelet's part therein were not included.

We are also assisted by the return of the members of our staff, Messrs. Porter and Pratley. Mr. Porter had been in charge of the design of the details for the Board of Engineers and, while the make-up of the members and the details finally used differed considerably from those of the official designs, the familiarity with these large and unusual details acquired by Mr. Porter during his service with the Board, no doubt enabled him to lay out this work much more rapidly than if it had been new to him. Mr. Pratley too, had much experience in the intricate calculations of the secondary stresses for the Board's designs, and was thus enabled to encompass a very large amount of work in calculating our designs, and because of the experience of these two men with the Board of Engineers our work was very much lightened.

The specifications issued, fixed the distance centre to centre of piers, 1758 feet; a clear head room of 150 feet for a distance of at least 600 feet in the centre of the span; the elevation of grade; the maximum height of steel work over the pier, 290 feet; the cross section of the roadway and the width, centre to centre of trusses, 88 feet; the loading and the unit stresses.

The requirements for material, testing and workmanship called for the very highest standard.

It was required that tenders on Contractors' designs be accompanied by complete stress sheets, deformation diagrams, details and make-up of all members, splices, lacing, and all information necessary to judge the adequacy and agreement with the specification of the proposed design.

The loading and unit stresses were briefly as follows:—

LIVE LOADS:

Trusses: Railway Load—on one or two tracks, 5000 lbs. per lin. ft. with 2-Cooper's E-50 Engines placed to give maximum stress.

Highway Load — 920 lbs. per lin. ft. of each roadway for trusses.

Floorbeams & Stringers and Members receiving Maximum Stress for a length of moving load covering 2 panels or less...............	100 lbs. per sq. ft. or 4,000 per lin. ft. of bridge. 2-53 Ton cars each 60 ft. long and 12 ft. wide on each track. A concentrated load of 24,000 lbs. 2 axles 10 feet apart.

DEAD LOAD:

Railway: Floor 670 lbs. per lin. ft. of each track.

Highway: Floor above stringers 2100 lbs. per lin. ft. each roadway. Snow load of 1500 lbs. per lin. ft. of bridge.

WIND LOAD: 30 lbs. per sq. ft. on surface of two trusses and train 14 feet high.

TRACTION: 750 lbs. per lin. ft. of one track.

UNIT STRESSES (CARBON STEEL)

	Live Load	Dead Load & Snow	All Coexisting stresses except secondary strains	All Coexisting stresses includ ing secondary Strains
	lbs. per sq. in.	lbs. per sq. in.	lbs. per sq. in.	lbs. per sq. in.
Tension Main Trusses.......	10,000	20,000	20,000	22,000
Suspenders and Members liable to sudden loading............	7,000	14,000	14'000	15,000
Railway Stringers...........	8,000	16,000	16,000	17,600
Floor beams & Highway stringers...........	9,000	18,000	18,000	19,800
Compression Members in Main Trusses..	$10{,}000-40\frac{l}{r}$	$20{,}000-80\frac{l}{r}$	$20{,}000-80\frac{l}{r}$	$22{,}000-88\frac{l}{r}$

NICKEL STEEL increase Units given for Carbon Steel as follows:

Tension.................................... 40%

Compression & Pins....................... 25%

PLATE XI

shows in outline all the designs submitted with the tenders. Official No
was the only design exhibited at the time tenders were called, but it w
later supplemented by Official No. 2 and tenders were asked on vario
modifications of these two designs.

The study of the Official Design and its erection had shown that
these great bridges the dominating factor of the design must be the pract
cability and safety of its erection, and this was emphasised in the develop
ment of our alternative designs. Economy of material is also an importan
factor because, in addition to its first cost, the effect of unnecessary weigh
may be cumulative, considerably increasing the sections of the heavies
members which are the measure of the shop equipment and of the erectio
appliances.

It was known that long panels with diagonals approximating an incli
nation of 45 degrees to the horizontal would give the most economic
form of Web System, but a diagonal inclined at 45 degrees would require
that the deep panels near the pier should be about 200 feet long. It was
seen at once that these long panels could not be constructed by the erection
methods we had decided to adopt, and that some compromise was
necessary either by shortening the panels or adopting a system of sub-
trussing.

A consideration of the Forth Bridge construction will perhaps assist in the further discussion of the other designs.

A length of the Forth Bridge equal to the projected Bridge at Quebec weighed about 35,000 tons—all carbon steel. The Official Design was estimated to weigh 72,700 tons, 52,000 tons of which was nickel steel. The Forth Bridge had successfully performed its duties for some 28 years and we were naturally not unmindful of this precedent—indeed, serious consideration was given to its design, and to the possibilities of adopting all or some of the special features giving it its remarkable economy in weight.

Referring to Drawings Nos. 2 and 7 it will be seen that the Forth Bridge differs in several particulars from the design of any of the other great bridges, and from any of the designs prepared for the Quebec Bridge.

The compression members are all circular in section and of large diameter. The circular section gives the largest radius of gyration for the metal employed, but this in itself was not an important factor as nearly every member in the bridge required sufficient area to give an effective radius with any of the ordinary forms of cross section. It results, however, in economy in the details because latticebars, tie-plates and other material which does not carry direct stress but is merely introduced to stiffen the rectangular member may be almost entirely omitted.

The trusses of all the bridges illustrated, including those of the designs submitted with tenders, lie in vertical parallel planes, while the trusses of the Forth Bridge are battered from a width of 120 feet at the shoes to 33 feet at the top.

There is important economy in this construction. The floor beams may be made only long enough to accommodate the traffic; the weight of the top lateral bracing and sway bracing is very much reduced and the section of the compression chords is also reduced. The total stress in the first panel of the bottom chord of the Quebec Bridge is about 15,000 tons, requiring a section of 1628 square inches; of this stress 4,000 tons is due to wind, requiring a section of 450 sq. inches, and had it been practicable to increase the width at the shoes to 120 feet, as in the Forth Bridge, the wind stress would have been reduced by about one-quarter and the section of the chord by 112 sq. inches.

The drawings do not show it, but if a horizontal section were taken at the floor level it would be seen that this section of all other bridges is rectangular, while in the Forth Bridge, the cantilevers taper towards each other [illegible]til at their ends they are only 32 feet wide. By making the suspended [illegible]an only wide enough to accommodate the traffic and for its own lateral stability considered as a simple span, much economy is realized due to the shortening of the lateral bracing and floor beams. As already pointed out, any saving in the weight of this portion of the bridge is multiplied several times in the weight of the cantilevers.

The diagonals have an effective inclination throughou[illegible] the trusses.

Members of such large dimensions as those used at t[illegible], could not be fabricated in shops at a distance, transported to the site [illegible] erected, and in the case of the Forth Bridge the manufacturing plant was a[illegible]ually built at the site of the Bridge.

Moreover, many of the details for connecting the circular compression web members were of necessity so complicated that it would have been next to impossible to manufacture them, except by the method of laying out each piece as the work was built up. The shapes and plates entering into the large members of the Forth Bridge were therefore marked off, bent, fitted and drilled or punched at the site, and the work was erected plate by plate and section by section—the whole system of laying off, fitting up, and rivetting, being closely analagous to steel shipbuilding.

This method of erection not calling for the handling of heavy pieces permitted each large member to be projected out plate by plate, carrying itself as a cantilever, until it reached a point of intersection with some other member, where they would give mutual support, or where in the case of a chord, a tie or strut could support the member from the intersection of the diagonals.

To carry out the work in the case of the Forth Bridge a large force men was required, the number being at times over four thousand. T work proceeded continuously throughout the year for over four yea There were plenty of men trained in steel shipbuilding on the Clyd and climatic conditions permitted the work to be carried on at all seaso of the year.

A sufficient number of skilled hand mechanics was not available Canada, it is doubtful if they could have been assembled even for continuo work, and when it is realized that erection work at Quebec could only b carried on for some six months in the year, it will be seen that it was qui impracticable to adopt this type of construction.

We therefore decided early that the only practical method of erectin the bridge in a reasonable time would be so to design it that the field forc would be a minimum. This necessitated manufacturing the members int as large pieces as could be transported and erected, and in order that thes pieces might be manufactured economically it was necessary to instal special plant capable of handling unusually large members, and requirin special machinery and mechanical appliances.

The use of this machinery also reduced the manufacturing force to the smallest possible limit. Our shop forces at no time exceeded five hundred men, and our field force working at the same time on the cantilever arms on both sides of the River, and on the erection of the suspended span at Sillery Cove, did not exceed, exclusive of field painters, 500 men, or about 1,000 all told for shop and erection. The average field force when only one cantilever was being erected, or before the erection of the suspended span, was very much less.

Although it was thus found impracticable to adopt circular compression sections or the field construction of the Forth Bridge, consideration was given to adapting the method of construction we proposed to battered trusses and to vertical trusses tapering in plan.

It will be realized from a consideration of the erection traveller and erection methods discussed, that members of the dimensions actually handled, or of any practicable size, could not be erected in an inclined plane without a very large amount of temporary supporting material, and that the field connections of inclined members would be most difficult.

With regard to the plan of vertical trusses with cantilever arms tapering in a horizontal plane, this also had ultimately to be abandoned on account of the difficulties of erection. A top chord traveller would have been impossible, due to the constant change of gauge of the tracks on which it would run and a through traveller could not be made wide enough for stability. Indeed, the only way of erecting either plan seemed to be by means of an outside traveller supported on temporary cantilever beams such as was used on the Phoenix Bridge. A few preliminary

sketches and estimates quickly demonstrated that the cost of erection and temporary material would far out-weigh the saving in weight to be gained by the narrower centre span. Even had this not been the case, the bent detail for connecting the members where the planes of the tapered trusses intersected, offered many difficulties, and it is doubtful if it could have been made with sufficient accuracy to insure the calculated distribution of stress throughout the intersecting members.

In a cantilever bridge of large dimensions the suspended span is the first element to be considered, as it may be designed and estimated without regard to the rest of the structure except as to its width, while the weight of the suspended span and the method of its erection must be considered in designing the trusses of the cantilever and anchor arms.

There was sufficient data on long simple spans to permit an economical arrangement of panels, outline of trusses and form of bracing to be readily chosen, and little trial designing was necessary here. It was desirable, however, to determine the method of placing the suspended span in position before fixing upon its length, its depth at the ends of the trusses or the outline of the trusses. If it were determined to erect it on falsework and float it into position, the depth at the ends would be kept low enough to give only sufficient head room for efficient portal bracing, so that the form of the chords might approximate to a parabola which was known by experience to give much the most economical outline for very long simple spans. If it were to be erected by cantilevering out from each side and joining in the centre, the end depth would of necessity be increased to something approximating the depth at the centre, otherwise it would be difficult to provide material for the heavy moment caused by the weight of the cantilevered portion, together with the weight of the erection travellers and material at the ends of the half span cantilevers, when the final connection was being made.

Even with the greatest practicable depth at the ends of the trusses, the cantilevering method of erecting the centre span called for a very considerable increase of material in the cantilevers and the span to provide for the erection stresses, and this in itself seemed a sufficient reason for early determining to float the centre span into position, thus permitting the most economical outline to be adopted.

There were, however, other considerations in addition to that of economy leading to this conclusion. When a suspended span is cantilevered out from each side, in order that the connections at the centre which convert it to an ordinary span may be made, the two halves must meet exactly in the centre and the upper and lower chords must be in alignment both vertically and horizontally. The length of the steel work varies with the temperature and from change of load, and the horizontal alignment may also vary from changes in temperature and wind loads. Adjustments must therefore be provided where the span connects to the cantilever in

order that these conditions may be met. Satisfactory appliances for the adjustments have been devised, and successfully used on all the gre cantilever bridges heretofore built, the centre spans of these having be erected by cantilevering, but in some instances the final connection at t centre of the span was only made after great trouble and with some ris The element of danger has lain in the difficulty of dealing sufficient rapidly with the existing heavy forces, when making the delicate adju ments necessary to follow the changes in length and alignment that tal place through the variations of temperature. Furthermore, when the centre connection is made, if the chord connections to the cantilever arr are not immediately released a change of temperature may cause the centr span to so connect up the cantilevers as to complete an arch from pie to pier, or at the other extreme to develop an undue amount of tensio in the top chords. Tentative designs for the adjustment mechanism required for the very heavy stresses found here, showed that it would b cumbersome and costly.

Having decided to float the centre span into position its length became a question of balancing; the increased difficulty of floating and placing a very long span against the saving in weight that might be effected by approaching the theoretical length for economy.

Theoretically, the least weight of material in the finished structure would result from making the suspended span about 1100 feet long, but this is a quite impracticable length for the span itself, and 668 feet was the longest simple span hitherto built. The Official Design divided the total span into three equal portions, making the cantilever arms and the suspended span each 586 feet. This arrangement was adopted as it fairly balanced the above considerations, and it facilitated the use of the official data in the new designs.

The types of trusses form which to choose may be enumerated as follows:

(1) The Official Design, Figures 1 and 2, Plate XI a Warren truss with each main panel divided in two and floor stringers of moderate length, forcing an uneconomical inclination to the diagonals.

(2) "M" Design, Figure 5; a Warren truss with each main panel divided in four, permitting the most economical inclination of the web members to be chosen while retaining short stringer lengths.

(3) A Warren truss having the diagonals at a favorable inclination without redundant members to support the top chord, each long panel being sub-divided in the middle to give a stringer length about twice that adopted for any of the other systems. See Figure 6.

(4) Double intersection trusses with and without sub-divisions.

(5) The "K" form of bracing.

The Phoenix Bridge with all diagonals in tension and sub-divided to give two stringer panels in each main panel, gained a favorable inclination of the diagonals and an economical from through the curvature of the bottom chord, but this form of truss was naturally not considered.

It was at once apparent that type No. 3 would give the least weight in the finished structure and have a pleasing appearance, but unless it were fabricated and erected in the same manner as the Forth Bridge, there seemed so many obstacles in the way of erecting it with safety and at reasonable expense, that merely outline sketches were prepared and serious consideration was not given to this type.

Some preliminary designs were made for double intersection trusses, but although the stresses and sections of the web members are only half those of the single intersection and the web members thus lighter and easier to handle, there seemed no way of getting favorable inclination of the web members without encountering similar erection difficulties to those met with in the single intersection design. A tentative design had been made by the Board on this system, but Mr. Vautelet had not found advantages in it to compensate for the stresses being statically indeterminate, and he was strongly opposed to its use.

Mr. Johnson and I early pinned our faith to the "K" form of bracing, but some of our associates wished to compare the merits of a bridge with a more conventional and more economical arrangement of bracing by preparing a complete design. Mr. Emil Larsson, Assistant Chief Engineer of the American Bridge Company, was asked to make this design, which is illustrated on Plate XI as "M-N".

Anticipating the more detailed discussion of the "M" design which will follow, it was at once realized that while it would make a very economical arrangement of the members actually carrying stresses there would be objections to its employment, in that the sub-truss system would probably introduce heavy secondary stresses from local panel loads; the top chord must be supported from the panel points as in the Official Design by long vertical members not useful for carrying live loads. During erection, very heavy temporary members would be necessary to hold up the inclined struts and the bottom chords until the main panel points were reached.

PLATE XII

is a general elevation of the "K" truss design submitted with the tenders.

The "K" form of bracing is quite different from the trusses of any of the other great cantilever bridges or from any form of trussing that had been previously illustrated in British or American practice.

It may be likened to a double intersection Warren truss with ever half panel reversed, and a vertical member inserted, thus retaining th advantages of the double intersection truss in halving the shears betwee two members in each panel. It is statically determinate as to stresse the shear being positively divided at each panel point, whereas in th double intersection truss, however great the care in calculating the stresses one system or the other may accumulate more than its share of stres due to errors in manufacture and erection. The top chord length o the double intersection truss is halved without redundant members It will be demonstrated later that no temporary work is required in erectio and that it is much the safest and easiest form of trussing to erect.

Most of the difficulties anticipated in manufacturing and erecting the Official Design had their principal source in the steep and uneconomical inclination of the web members, the large section of these members, and the necessity of either having the erection traveller extend out over two panels or having a very large quantity of temporary holding-up material, expensive to supply and very difficult to place and adjust with the required nicety. With the "K" form of truss the main panel could be halved for the same angle of web bracing; or, conversely, for the full panel the angle of the bracing would be nearly twice as favorable. The early sketches were for the full panel of 84 feet, and for several lengths down to panels of 65 feet long, all to be erected by a top chord traveller.

The "K" truss design lent itself to the use of a top chord traveller, the normal stresses from live and dead load in the vertical posts requiring sufficient section to carry the erection stresses imposed by the traveller, and the cost of the erection equipment for this arrangement would have been comparatively small. The long panels, however, required such heavy floor beams that there were difficulties in the way of manufacturing, transportating and erecting them. The stringers were also heavy, and the general arrangement did not realize the economy expected. Moreover, the risk of using a top chord traveller with two panels reach was not in any way abated.

Deformation diagrams were made for the sub-panel now used. It was found that this form of sub-trussing did not introduce objectionable secondary stresses, and did not interfere with fully splicing the joints of the compression members as the erection proceeded. Altogether this arrangement had everything to recommend it, and it was adopted.

The trial of the sub-panel was initiated in the endeavour to use a through traveller which would avoid the risks attendant on the use of the top chord traveller. The through traveller is necessarily much heavier than the top traveller, and one to reach 84 feet or even 65 feet, was found quite impracticable. There was no doubt, however, that a traveller could easily be designed of moderate weight that would place members

of the size to be handled at a distance of about 42 feet from its forward point of support. The early designs for this traveller were made with four swinging booms each capable of lifting the heaviest member to be handled and swung by guy-tackles from a horizontal frame-work attached to the top of the traveller. As the erection was worked out in detail, it was found that the use of the booms would not be entirely satisfactory, and we were led to adopt the travelling cranes, notwithstanding that this arrangement added much to the cost of the equipemnt.

Touching on the outline of the whole structure,—the length and outline of the centre span had early been determined. The maximum height of the cantilever arms over the main piers was fixed by the specification. A lower height was undesirable, because even if it resulted in economy of total material, it would require heavier chord members near the piers and these large members, as already pointed out, fixed the size and capacity of the erection traveller and lifting devices as well as the cranes and many shop tools.

Economy demanded that the end height of the cantilevers should be as low as practicable provided the end post from which the centre span is suspended should have an economical inclination approaching 45 degrees. The outline of the bottom chord of the bridge was also fixed by the specified height of the central portion to give clearance for navigation, and the requirement of straight chords between this central portion and the main shoes.

For the sake of appearance, repetition of shop work and other reasons, it was desirable that the anchor and cantilever arms should have the same panel lengths and form of trussing, but it was found economical to shorten the anchor arms by one full panel. There would probably have been economy of material in a still shorter anchor arm, but it would have increased the already heavy load on the main piers, 502 feet was the shortest anchor arm of the Official Designs, and a shorter design was not tried.

PLATES XIII and XIV

show the general arrangement of the traveller used for erecting the Quebec Bridge. It was equipped with two travelling cranes, each having two trolleys of 60 tons capacity, that could be run out far enough to lift the outside members of the trusses, and auxiliary hoists at the ends of each crane of 7 tons capacity. The four derrick booms each had a capacity of 15 tons. The total weight of the traveller and rigging was about 940 tons, and with both cranes run to the rear position for moving, the load was almost equally distributed on the front and rear trucks. When lifting the maximum loads the reaction at the front points of support was about 1300 tons.

The top of the steel work of the traveller was about 210 feet above the rail on which it ran. The crane runways were about 140 feet and pe[illegible]ted a crane to work 40 feet in advance of the point of support.

The above short description of the traveller is given here to convey an idea of its size, and the importance of this portion of the field equipment. It will be fully described in the forthcoming paper.

After planning our erection for the "K" truss we found that this form of traveller could be advantageously applied to Official Design V and we estimated on using it for erecting that design.

Fig. 2.

shows the method proposed for using this traveller in erecting Official Design V. It will be seen that as the traveller is moved out from floor beam to floor beam, it is necessary to support it, and a portion of the bridge, by means of temporary ties until it has reached a position where it stands on the fourth floor beam away from the shoe where the unit of the main truss [illegible]ystem is completed, and the same condition obtains for the succeeding four floor beams. It will also be noted that until its rear portion has moved past the top of the inclined strut, the permanent sway bracing cannot be placed between the compression struts, and a large portion of the bridge is without transverse bracing above the floor system.

To understand this we shall have to refer to the system of wind bracing adopted. It has been usual to place a system of lateral bracing in the planes of both the top and bottom chords to provide for wind loads and vibrations, and to keep the compression members in line. The stresses, or part of then, that go through the lateral bracing in the plane of the top chord are then considered as being carried to the piers by a system of sway bracing between the main posts'

This arrangement results in some ambiguity of stress, and the Board properly decided that the more correct and economical method would be by means of sway bracing between the compression members, to transfer all horizontal loads to the bottom chord, and thus carry all the horizontal loads through the lateral bracing of the bottom chord to the piers where they must eventually be resisted.

This system of top lateral and sway bracing, excellent in itself, was not well adapted to a through traveller as in several positions of the traveller the sway bracing would have to be omitted from between the inclined struts until the traveller had moved out clear of them and there would thus be two panels of truss without permanent lateral bracing.

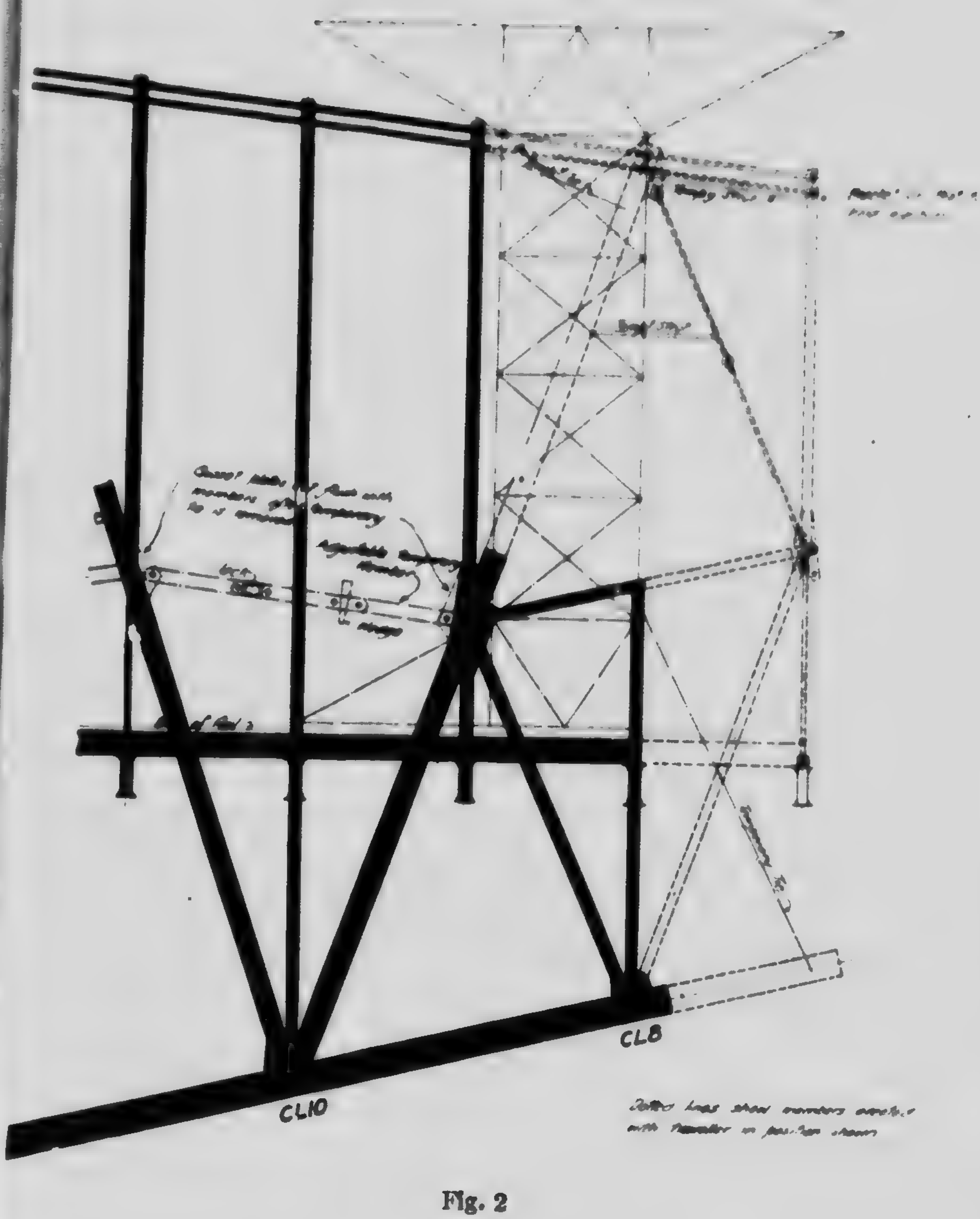

Fig. 2

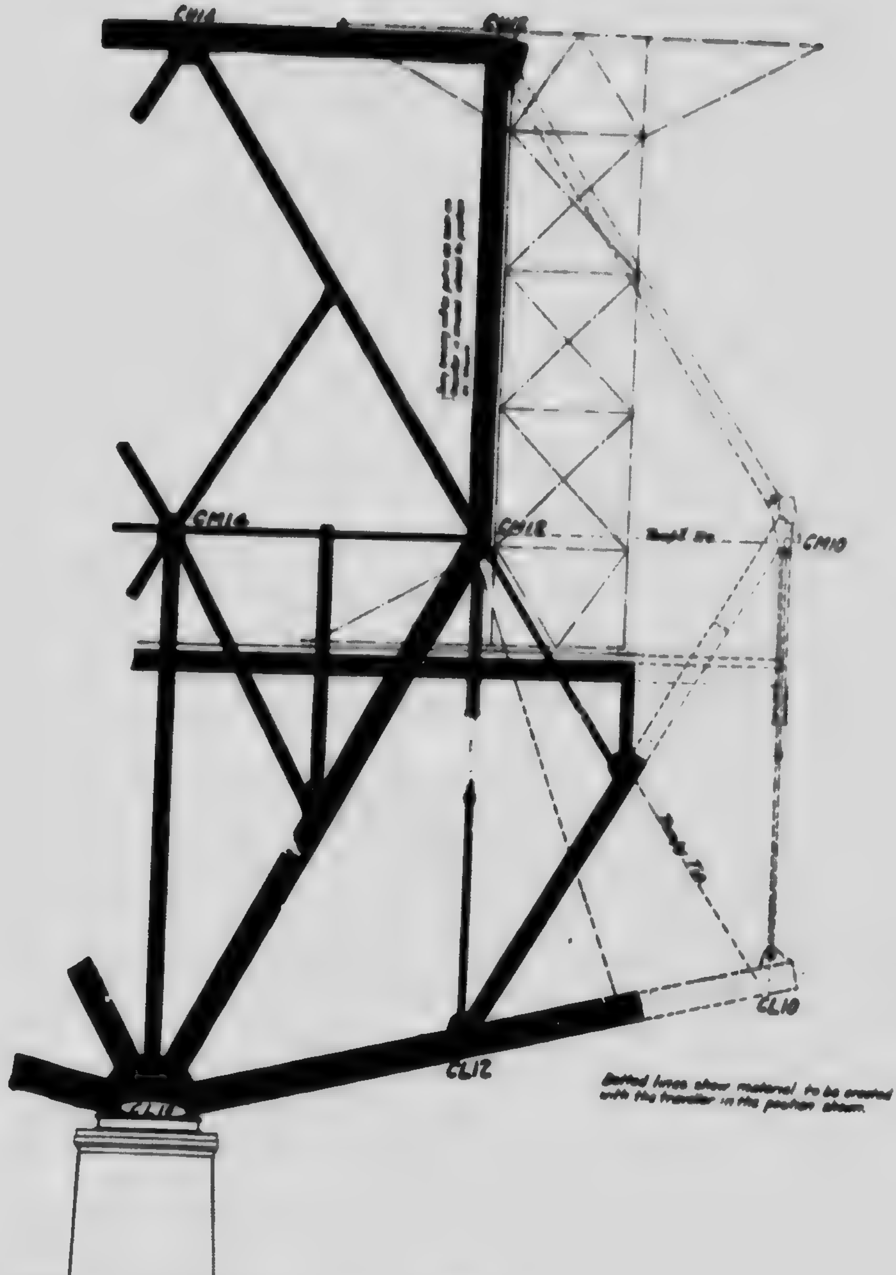

Fig. 3

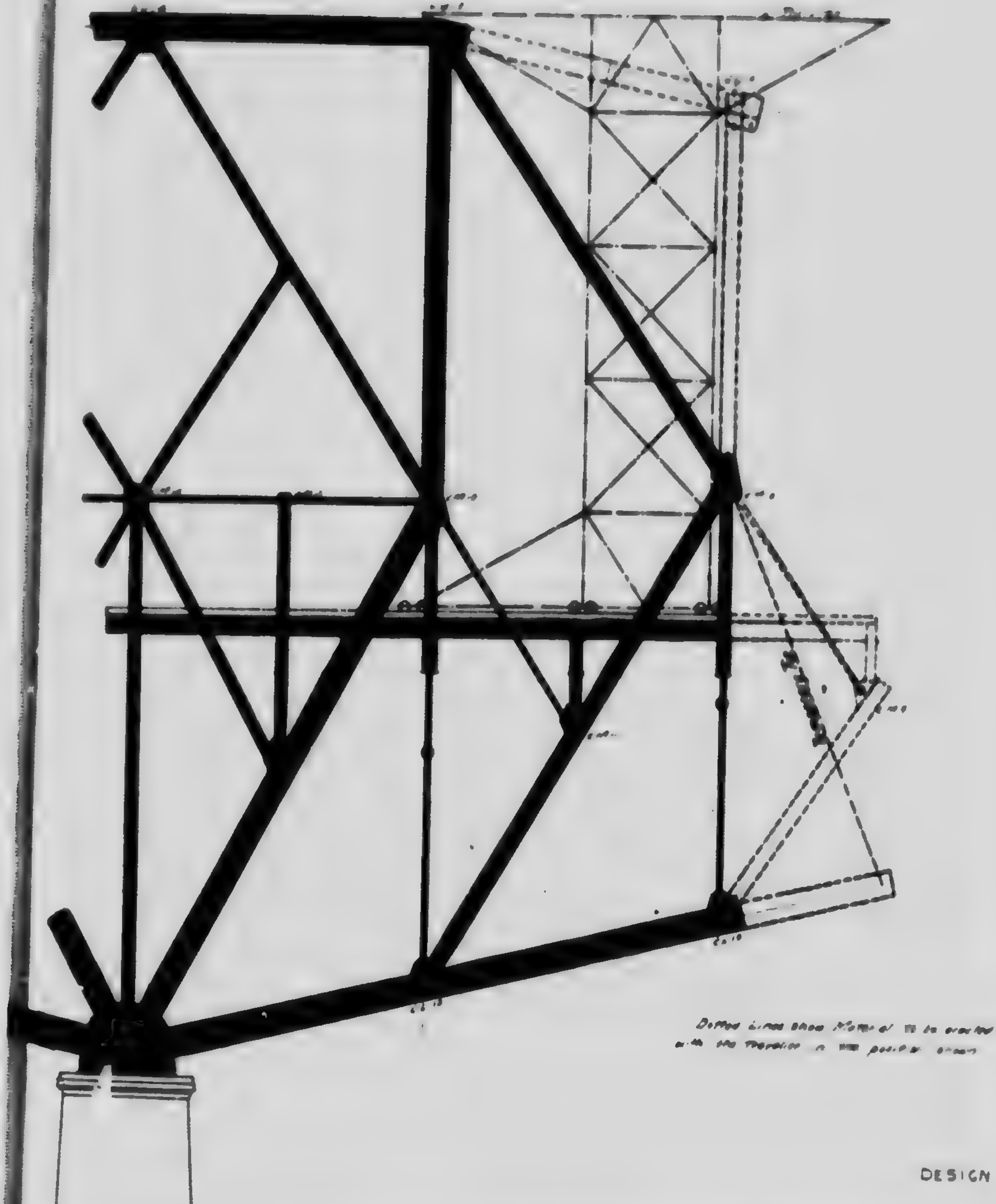
Dotted Lines show Material to be erected with the Traveller in the position shown.
DESIGN

Fig. 4

Figs 3 and 4.

show the traveller when erecting the "K" truss. It will be noted that in all positions of the traveller, each floor beam when placed is carried by the permanent construction of the bridge and no temporary work is required except for holding up the bottom chord, and that, only because it is convenient to place the lower members first. If the diagonal members were placed first they would interfere with the tackle necessary for placing the lower chord.

Another advantage in the erection of the "K" truss is that the lateral and sway bracing can be placed between the vertical posts as soon as the traveller has moved past one of these posts and, moreover, can be erected by the traveller itself, so that the structure as it is erected is braced and securely held against all lateral forces by the permanent construction.

Fig. 5.

is a deformation diagram for both anchor and cantilever arms showing the framed form of the trusses. It is shown here to illustrate the absence of sharp bends and the generally uniform deformation resulting from the "K" form of bracing, an advantage that had not altogether been anticipated when the designs were started. It was early demonstrated, however, as it was necessary to make deformation diagrams for each stage of the erection, to determine whether there would be bends or undue distortions during erection. As a further illustration of the uniform deformation of the "K" truss,—

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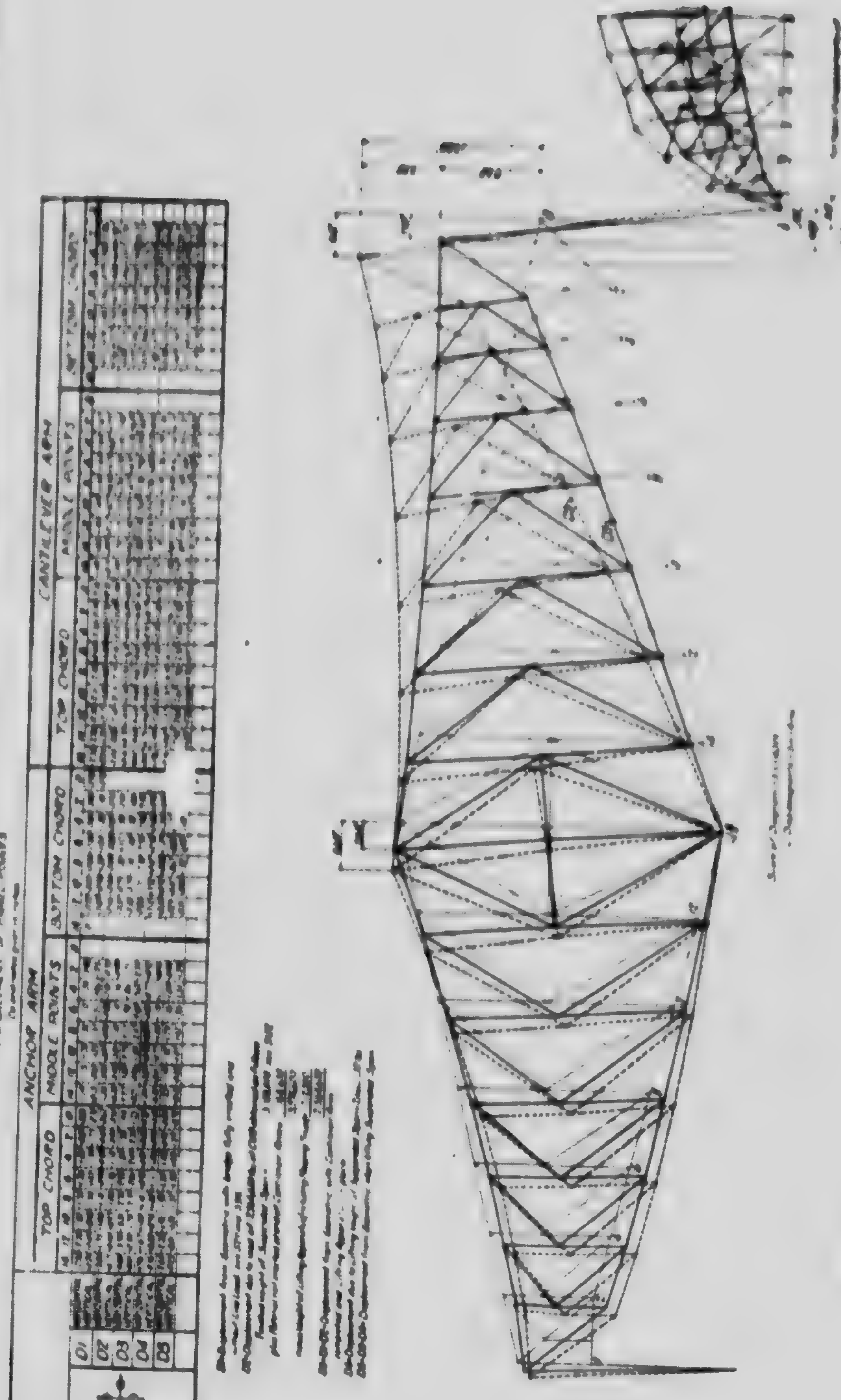


Fig. 5

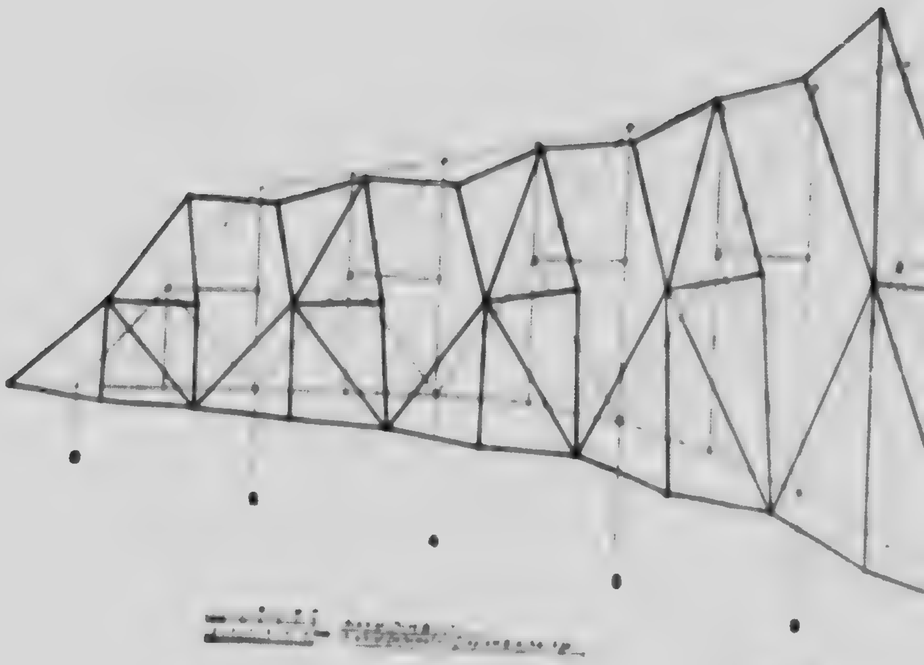


Fig. 6

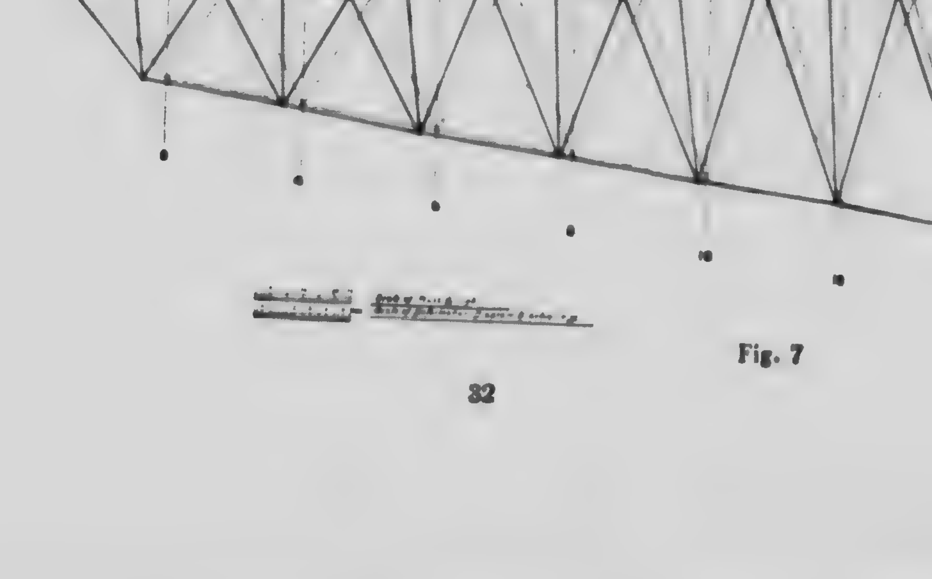


Fig. 7

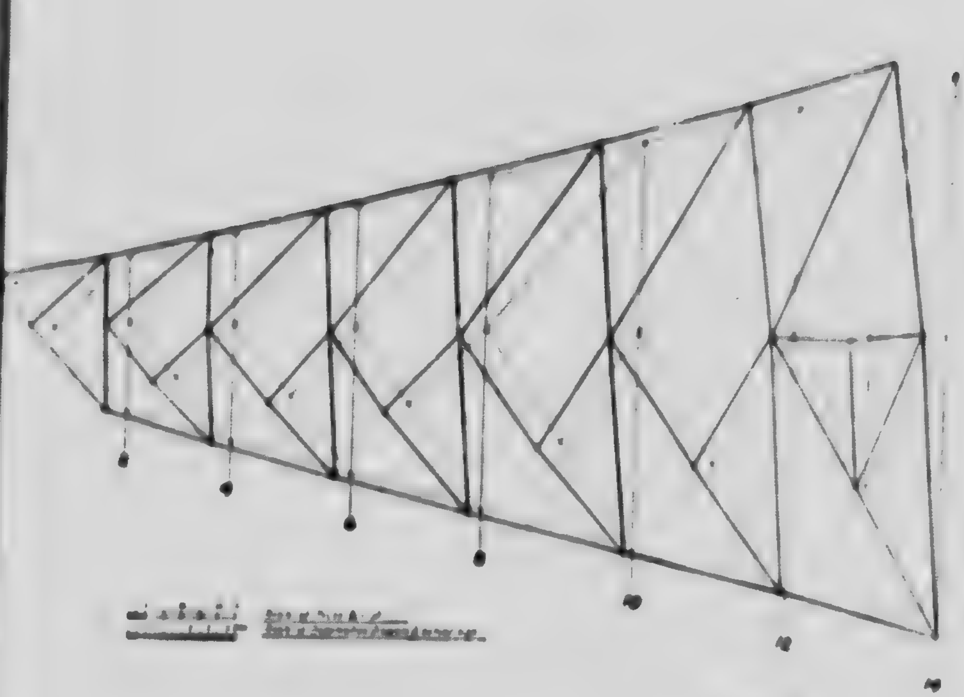


Fig. 8

Fig. 6, 7 and 8

show the anchor arms of the Phœnix truss, the Official Design and the "K" truss. The distortions of the Phœnix truss are very marked, particularly those of the bottom chord. The lower chord of the Official Design has a uniform deflection, the long diagonal members are kept straight through the adjustment of the upper chord of the sub-truss before referred to, but there are marked kinks in the upper chord where the long vertical redundant members push it out of line.

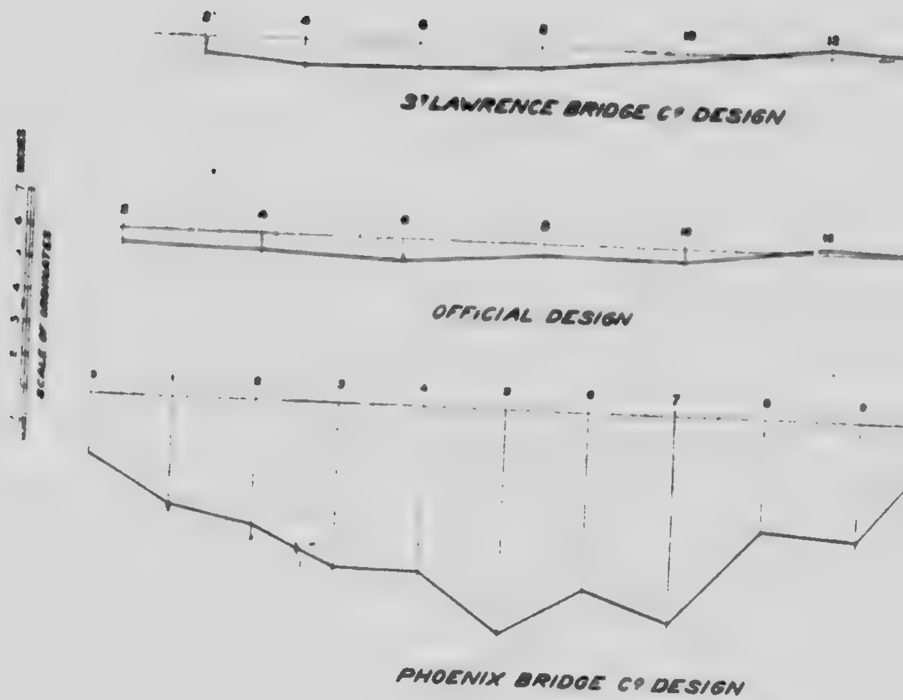


Fig. 9

Fig. 9.

shows a comparison of the bottom chord deflections under dead plus live load for the three designs.

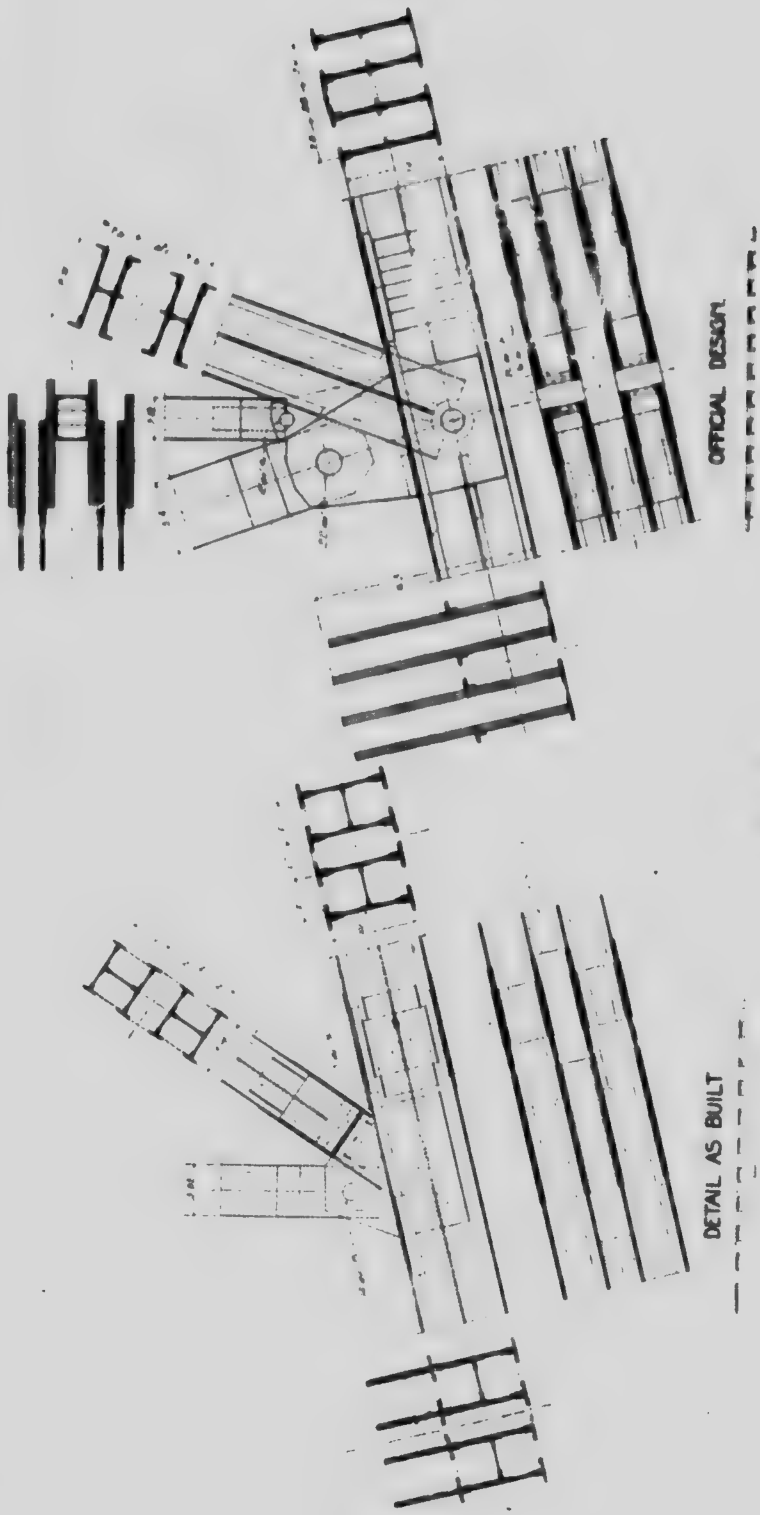


Fig. 10

Fig. 10.

shows the connections of the web members of the bottom chor at the first main panel point of the Official Design, and the connection for a corresponding panel point of the "K" design. The Diagram als shows sections of the chords and compression diagonals of the Officia Design and the "K" design.

There are many objectionable features in the details of the connections of the Official Design. The main compression members must be forked for a long distance in order to connect on pins in the center of the chord. The clearances of the long forked ends were necessarily small and called for great care in manufacture to permit the field connections to be made.

The pins on which these members abut carry heavy moments, and are very large. The plates to which the tension diagonals connect are very deep, and the pin holes a long distance from the bottom of the chord.

The pins in the centre of the chord prevented the use of centre diaphragms, and to provide the necessary lateral stability for each pair of webs they were connected by cover plates on the bottom flange with a row of lattice bars at the centre of the section and another row on the top flange. The section of the cover plate on the bottom was compensated for on top by narrow plates rivetted to the flange of each web girder. While the area of the metal was thus properly distributed about the centre line, the section was unsymmetrical and required for lateral stability an unduly large proportion of lattice bars not carrying stress.

In the "K" design the stresses in the web members are practically halved, as there are two diagonals in each panel to carry the vertical shear. Owing to this and the more favorable angle of inclination of the diagonals, it was possible to make all connections on pins outside the chords, and to keep the webs of the compression diagonals in the same plane as the connection plates, thus relieving the pins of all moments and making it necessary to provide pins only of sufficient size to carry the direct bearing.

There were no pin holes in the centre line of the chord, and centre diaphragms were carried through between the outside webs, making an exactly symmetrical section that tested well, was easy to manufacture and handle, and considerably more economical in the weight of details than the Official Design.

There was another advantage in the chords of the "K" truss not apparent from the sketches—namely, in the more uniform increment of load received at each panel point from the cantilever end to the shoe, permitting a more gradual increase of section, and for this reason better splices and better details throughout. This will be better understood when it is considered that in the Official Design there is no change in chord stress, due to

vertical loads, from the shoe to the first main panel point, approximately 170 feet from the shoe; and, similarly, there is no change in the chord stress from this panel to the next main panel point, 170 feet further on. In the "K" design the main panel points occur at half the distance of those in the Official Design, and the increments of stress are correspondingly less and more often applied, so that the increase in the chord section at any panel point of the "K" design is only about half the increase at the main panel points of the Official Design.

Reference has been made to the large number of vertical members in the Official Design, and in our "M" design, introduced simply for carrying the weight of the top chord. It will be seen that in the "K" truss every member of the truss carries its proportion of live as well as dead load and there are no redundant members. As shown on the deflection diagrams of the Official and Phoenix designs, the redundant members cause considerable distortions of the frame, and the omission of these members which have no part in carrying the live load of the structure results in important economy.

The question of appearance must be largely one of individual preference, and it is perhaps difficult to compare designs shown only on a flat elevation. The redundant members in the Official Design and the varying inclination of the chords of the sub-trusses and stiffening struts, seemed to us to detract from its appearance. We think the symmetry and evident purpose of every member in the "K" design gives it a certain appearance of fitness and dignity.

When referring to the webs of the compression diagonals being practically in the same plane as the webs of the chords, attention was not called to the fact that this gave extra spacing between the outside webs, permitting each web to have double angles, thus making it symmetrical and of stronger section than the channel form of web, and moreover, giving a much larger and sufficient radius of gyration for the columns as they were shipped from the shop and erected, making it unnecessary to place much dependence on the lacing and tie-plates field rivetted between these members after their erection.

Another important consideration from the purchasers view point is that, with the construction adopted, access for inspection and painting can be obtained to all parts of the interior of the chords and large compression members; whereas, with the long forked ends proposed for the Official Design there were of necessity many narrow spaces that could not be reached after the Bridge was erected.

The other comparisons have principally to do with the cost of material and operations in detail and cannot easily be set forth. We, however, estimated there were many economies in the "K" design.

PLATE XV

shows a general elevation of the "M" design. It will be seen that th design is pleasing in appearance; the detailed estimates of weight wer as expected, considerably lower than for any of the other designs, bein only about 85% of the next lightest, and it may be asked why we did no make a stronger effort to obtain the contract on this design.

PLATE XVI

is a portion of the "M" design to a larger scale, showing the general charac ter of the details and connections proposed.

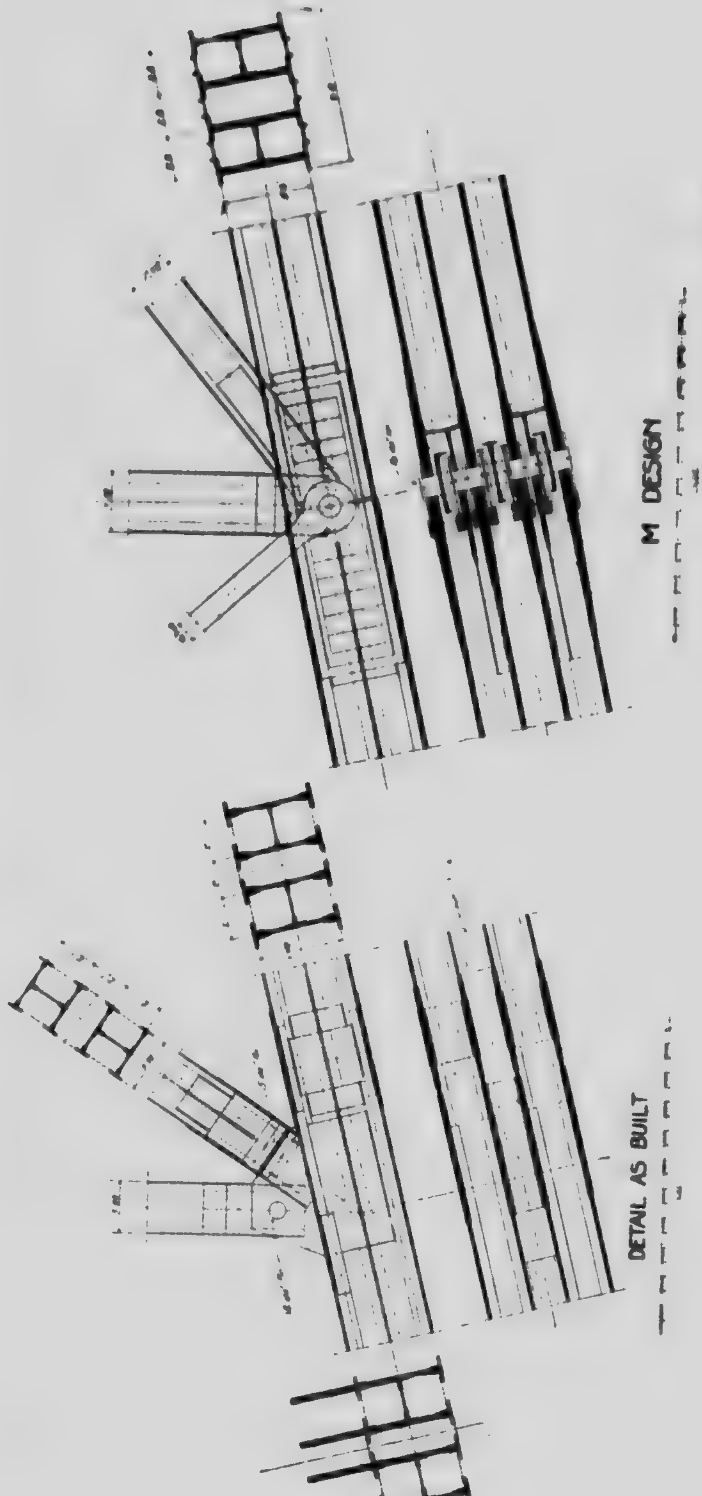


Fig. 11

Fig. 11.

shows the detail of the first main panel point in the bottom chord compared with the corresponding panel point of the "K" design.

In order to carry out the design logically and in such a manner that it could be erected safely, it was necessary to introduce pins at all panel points in the compression members and the use of pins introduced many objectionable features which Mr. Vautelet and the Board of Engineers had been at great pains to avoid in the Official Design, views in which we entirely concurred. When the design was started it was not known how these details would work out—indeed, as already pointed out, the details of any new design for a bridge of this magnitude could only be worked up by carrying the design through by the tedious method of working from the centre span and the cantilever ends back to the piers. Having carried the design to this stage of completion, we thought it worth while putting in a tender as a matter of record, although we did not in any way urge its claims.

It has been shown in discussing the open erection joints of the Phoenix truss, that a truss of this form could not be safely erected without providing for angular movement at every panel point of the compression member; hence the reason for the pins. It was doubtful, however, if ordinary pins could perform the intended function of permitting this angular movement because the movement is gradual as the load comes on and on some pins, when the cantilever arm has been extended out some distance, must be made under heavy stress. If movement were to be assured, it is probable, that the unit pressure on the pins would have to be much less than the bearing pressure allowed for the transmission of axial stress. This is provided for by large sleeves at the shoe-pins of the Official Design and of the Bridge as built.

It will be seen that to provide pin bearing even for the stresses permitted by the specification, it was necessary to build up the chord webs of the "M" design to great thickness, and that in some instances all the reinforcing was placed on the outside of the web in order to make room for the long fork ends of the members connecting on the pin. This, which may be termed eccentric reinforcing, is of doubtful value even if the inner pin plates are made sufficiently long to assure that the rivets all act in shear.

The necessity of stopping the diaphragms towards the ends of the chord joints, thus making fork ends on the chords as well as on the web members, also seemed a very undesirable detail.

Added to the above considerations, we concurred with the Board of Engineers that these very large compression members should be as nearly as practicable continuous from end to end, so that the distribution of stress throughout the member should not be disturbed, and that the stresses would continue without being deflected past the splices and past the panel points where the member would receive a fresh increment of stress to be provided for by additional material. With pin joints it takes a great deal

of reinforcing material to concentrate upon the pins the compression stresses carried by the outer edges of the web and the flanges, and these stresses must be again distributed over the section of the member after the joint is passed.

A question that may be asked is why the "K" bracing was not carried through the suspended truss, so many advantages having been found in its use in the cantilever arms and the suspended truss itself being of such very considerable length. "K" bracing was designed for the suspended span, and had we determined to erect this as a cantilever it would no doubt have been used, but it does not lend itself well to the parabolic form of top chord or to the low end heights which it seemed economical to use for this truss and, considering the method of its erection and the deformation diagrams, there seemed no objection to the form of truss used.

The discussion heretofore has referred to the designs submitted with the tenders and principally to Design "B" of the St. Lawrence Bridge Company recommended by the Advisory Board of Engineers for acceptance by the Government. That Design was for the specified lengh of 1758 feet centre to centre of piers, and its outline elevation necessarily differs from that of the present Bridge which has a span of 1800 feet centre to centre of piers.

PLATE XVII

shows the general elevation of the Bridge as built.

The masonry of the old Phoenix Bridge was left in perfect condition after the accident, but the piers were too short to carry the projected span, it having been determined to increase the 67 ft. width of the Phoenix Bridge to 88 feet in the new Bridge. Mr. Vautelet had intented to make use of the old foundation of the pier on the South side, increasing its dimensions, however, both in width and length by sinking new caissons alongside, and to build a new pier on the North side of the River clear of and to the South of the old pier—the plan working out to a distance of 1758 feet centre to centre of piers with the centre line about 15 feet down stream from the centre line of the old bridge.

It was finally concluded that there might be great difficulty in sinking new caissons alongside the old foundations of the South Pier—indeed, it was thought by some that the proposed construction was quite impracticable. After Mr. Vautelet had resigned, the Board, in conjucntion with the Advisory Engineers, recognizing the risk of at least serious delay from this cause, made a number of studies for using the foundations of the old piers. These studies were based on building up new masonry on the old foundations of the maximum length that the foundation caissons would permit, and reducing the weight upon the foundations by carrying a portion of the load on new piers to be built to the South of each of the present piers, thus giving four points of support for each cantilever instead of two as in the Official and the present design. Studies were also made for sinking three new pedestals on each side of the River and making use of the old foundations for the fourth pedestal.

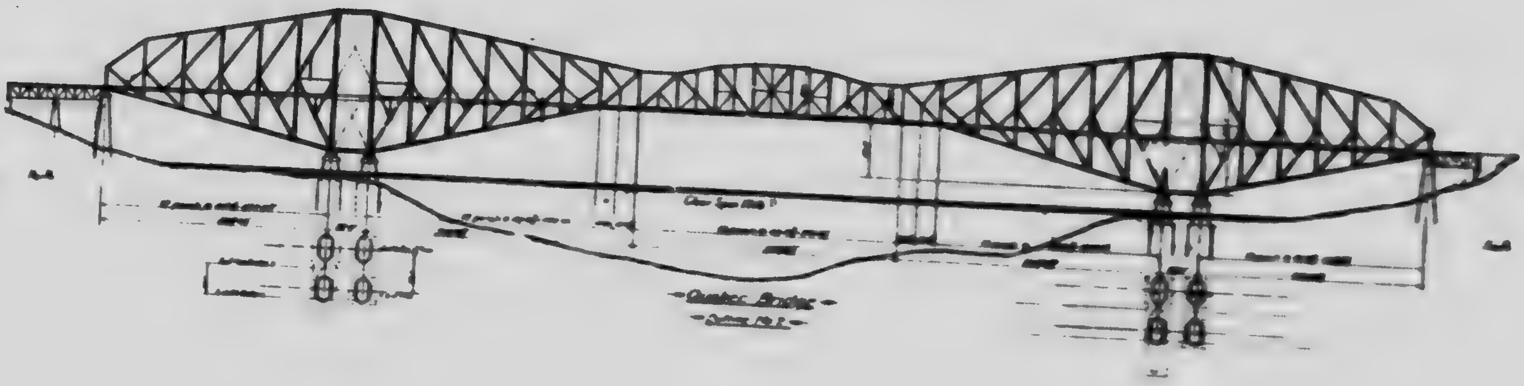


Fig. 12

Fig. 12.

shows the proposed design of the superstructure for one of these plans.

It was not found practicable to make the piers long enough for the width of bridge without too great intensity of pressure on the ends of the caissons, and the three new pedestals did not promise much economy. The plan to make use of any part of the old masonry or the old foundations was finally abandoned, and the Board recommended sinking new piers to the South and entirely clear of the old foundations but on the same centre line, thus restoring the span to 1800 feet.

In adjusting the "B" design to the new length of span the Advisory Board considered that the Bridge would have a better appearance if the low panels towards the ends of the cantilevers and of the suspended span were made shorter, in order that the inclination of the web members might be kept at about the same angle. In conference with the Board, the length of the suspended span was fixed at 640 feet and the cantilever arms at 580 feet. The span was divided into four more panels, one being added to each cantilever arm and two in the suspended span.

Mr. Vautelet had specified that the height of the steel work above the centre pier should not exceed 290 feet and had refused to permit any greater height. The Advisory Board saw no reason for this limitation and the inclined upper chords of the anchor and cantilever arms were continued until they intersected over the centre pier, the Board considering that this arrangement gave a better appearance, as it undoubtedly does on paper, than the two panels of flat chord over the piers. The intersection of the chords at this point made it necessary to extend the vertical posts to carry the shears from these chords and, having adopted this vertical post to omit the horizontal tie holding the heads of the first compression web members and replace it by inclined tie bars, thus carrying the "K" system of bracing from the inclined posts at the end of both cantilever and anchor arms throughout all the panels to the main pier.

In making our competitive designs we were necessarily governed by considerations of economy, and endeavoured to adopt an outline that would result in the lightest structure consistent with the specifications and the other requirements sought, including good appearance.

The increase in span called for a heavier structure and the additional panels, as well as the changes in the pier panel, and also made considerable additions to the weight, but the Board considered that in a monumental structure of this character the extra weight was justified by the improved appearance. Personally, I am not at all convinced that the more economical arrangement of the "B" design would not have given an equally good appearance in the completed structure had it been adopted.

PLATE XVIII

shows a cross section of the floor of the Official Design together with the live and dead loads above the railway stringers.

A cross section of the floor and the loads on which our Design "X" was based.

And a cross section of the floor of the bridge as built.

Throughout the consideration of the construction of the Official Design and of the alternative designs, we were faced with the problem of manufacturing and placing material having weight and dimensions far beyond any precedent, and it was constantly being forced on our attention that because of the span and the construction approaching the practicable limits it was of great importance to avoid any unnecessary weight either in live or in dead load.

A consideration of the heavy construction required to carry the highways specified, and the increase in the weight of the structure arising therefrom led us to make some approximate figures for a bridge to carry railway traffic only. These were so encouraging that they were followed up by complete designs omitting the highways, with the exception of a side-walk, which it was thought should be retained for the sake of inspection and such pedestrians as might wish to cross the bridge.

These designs were otherwise in strict accordance with the specification and were similar to the designs carrying highways. This plan saved 3100 lbs. per lin. ft. of superimposed load and 4,480 lbs. per lin. ft. in the weight of the roadway. The saving in the structure itself averaged 11,900 lbs of steel per lin. ft. and the finished structure was estimated to weigh only about 75% of that designed to carry the highways in accordance with the Board's specification.

Fig. 13.

shows sections of the bottom chords next the main pier of the Official Design, the "B" design and the "X" design without the highways.

In addition to the economy in weight to be realized, there were many good reasons for advocating this change of design, as it brought the size of the members within more practicable limits for manufacture and erection and the small interests to be served by the highways seemed out of all proportion to the additional cost, and the risk and difficulties in construction that the highways as designed entailed. After the Advisory Board had recommended the acceptance of our tender for Design "B", designed to carry the full highway and electric car loads as specified and had modified the span and outline as above noted, we pointed out to them the economy and advantages to be gained by omitting the highways and, when the final contract was drawn, the specification and plans accompanying it omitted the provision for these highways.

In preparing our alternative design without the highways, it was not considered advisable to risk undue criticism by making any further change than necessary and the highways were simply left off, leaving the tracks as spaced in the Official Design. It was manifest, however, that advantages would accrue by separating the tracks as widely as the clearance of the sway bracing and the torsion of the structure would permit, these advantages being reduced stresses in the floor beams, a better staying of the top flanges of the floor beams and easier provision for traction stresses. The Board of Engineers readily accepted this change and the tracks were moved out to a width of 32'6" centres.

During the discussion of this change Mr. Monsarrat advocated a further change to the form of floor system used under his direction by the C. P. Ry. on some of its high viaducts. This floor, which is shown by Figure 3 on Drawing No. 30 consists in placing each track in a through plate girder bridge with steel stringers and floors beams of its own, and with exceptionally well braced and reinforced top flanges, so that in the event of derailment, the derailed rolling stock would be kept in this trough and have no opportunity to plunge down and wreck other members of the bridge.

Another change from the accepted design was in the use of eye-bars for the top chord. We followed the Official Design in the use of carbon and nickel steel, and tendered on all of our designs in this way, in order that our tenders should not be set aside through not complying with the requirements. We also gave alternative tenders on what we considered more economical proportions of carbon steel, but we tried to avoid using carbon steel where its use could be criticised on account of adding unnecessary weight to the bridge. While we tendered in some of our designs on using nickel steel eye-bars, experiments up to that time had not shown these bars to be entirely reliable, the quotations for them were exceedingly high compared to other material, and in our "B" design we estimated on plate nickel steel chords with rivetted connections. Some of the members of the Board preferred eye-bar top chords, and after the contract was awarded we obtained quotations for carbon steel eye-bars, prepared estimates comparing the weights and cost of these with the material shown on the contract drawing, and under a supplementary contract substituted the eye-bar top chords in the final design of the bridge.

When discussing the changes in length of span and modifications of outline, the Advisory Board of Engineers also discussed some changes in the specification to bring its form more in accord with existing standard specifications without sensibly changing its requirements except, in the substitution of E-60 for E-50 locomotives; and when it was finally decided to build the bridge for railway traffic only, a new specification covering this condition and the other changes was drawn up.

The specifications finally issued by the Board of Engineers provided for a double railway track, but for no highways, except two sidewalks five feet wide. They will be given in full in the forthcoming paper.

The greatest care was taken to comply with the requirements of the specification, and to submit with our tenders all the information called for therein. Had there been no change in the length of span, our plans were ready for the preparation of shop drawings, but the changes already enumerated necessitated an entirely new set of computations and, while the details and the make-up of the members remained of the same type as originally designed, modifications were necessary in nearly every instance by reason of these changes.

Before tendering it was necessary, for purposes of estimating, to make approximate plans of shops and equipment, the erection travellers, falsework and other equipment required in the field, so that we might assure ourselves of the practicability of the work and make close estimates of the cost of the whole. These sketches were, however, not working drawings and as soon as the contract was let it was necessary to organise for carrying out the new design, the preparation of shop drawings, the manufacture and the erection of the structure. We felt that great perfection and a new standard of shop work would be required, that there was so much to be done, so many unusual problems to consider and so short a time in which to do it, that we must employ the best engineering and manufacturing advice that could be obtained.

ORGANIZATION OF ST LAWRENCE BRIDGE CO LTD

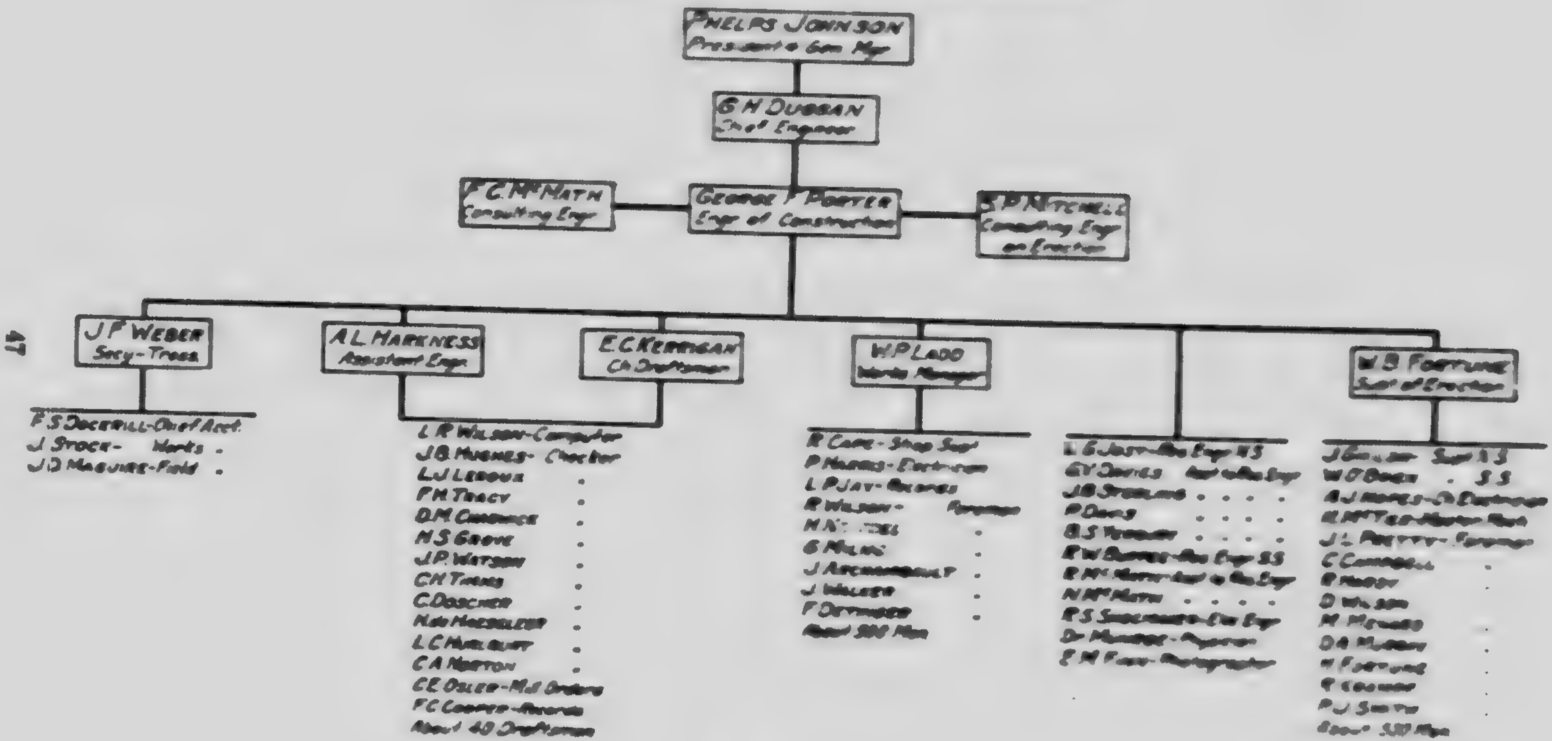


Fig. 14

Fig. 14.

shows the working organization of the St. Lawrence Bridge Company for the construction of the bridge.

Mr. Johnson as President kept an engineering as well as a business upervision of his charge.

Mr. McMath was available for consultation when required, all drawings were submitted to him and were examined by him personally. Mr. Willard Pope, Chief Engineer of the Canadian Bridge Company rendered much valuable assistance, personally checking all strain sheets.

Mr. Porter was put in immediate charge of the Engineering force as Construction Engineer. To him belongs a very large share of the credit for the conduct of the work. In addition to his assistance in preparing the details for our tender designs, he acted as Resident Engineer throughout, overseeing the preparation of shop drawings and the detailed design of erection equipment at Montreal during the Winter, and during the summer residing at the site in responsible charge of the field operations. From the outset, the work under his direction was carried on so satisfactorily that much more responsibility than generally falls to the position was placed upon him with the utmost confidence.

The Canadian Bridge Company released Mr. W. P. Ladd, then Superintendent of their shops at Walkerville, as Works Manager of the new shops at Rockfield and it became his duty to take charge of the layout, purchase of equipment, organization and, indeed, everything in connection with the manufacture of the bridge and the erection equipment. The excellence of the shop work and the wonderful precision in the lengths and fitting of the various members have been commented on by every engineer who has viewed the work. This excellence contributed much to the facility with which the bridge was put together in the field, and we all realise that the work of Mr. Ladd and his staff was one of the most important factors in the operations.

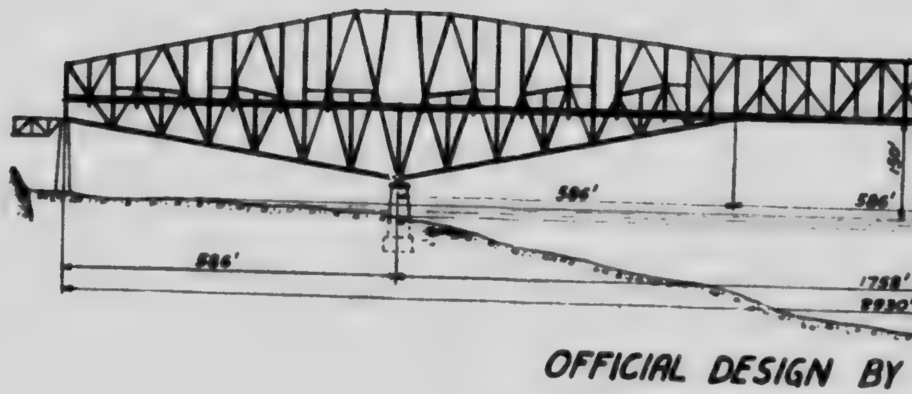
We felt that the erection, involving as it did the lifting and placing of the heaviest pieces heretofore handled, and by far the largest tonnage in one span, required the best experience in heavy work and the best expert advice that could be obtained, and we were fortunate in securing the services of Mr. S. P. Mitchell as Consulting Engineer on erection, he being considered best qualified to supply the experience and advice needed. Mr. Mitchell devoted a large portion of his time to the consideration of the erection equipemnt, secured for us the services of Mr. W. B. Fortune, our General Superintendent of Erection, and assisted in organizing the field force.

Where so many experienced men have been employed and given their best thought to the work over a period of about six years, it is impossible to particularize further, but it will no doubt be appreciated that many helpful su   stions and much assistance were received from the Assistant Engineer, Mr. Harkness, the Chief Draftsman Mr. Kerrigan and his staff, the Superintendents and the Inspectors both in the shop and in the field. The accurate work of Mr. Jost, Mr. Burpee and the field engineers should not be overlooked.

Reference must be made to our harmonious relations with the Board of Engineers and the assistance received through the hearty co-operation of its Members. The experience and judgment of Mr. Modjeski, Mr. Charles MacDonald and the Advisory Engineers when discussing the changes in plan was of great value, and it goes without saying that anything with which Mr. Schneider had to do must bear some evidence of his knowledge and ripe judgment in all matters pertaining to steel construction. Notwithstanding Clause 5 of the Specification which threw the entire responsibility of the design, material and construction upon the Contractor, the Board of Engineers organized an experienced and highly efficient staff of engineers and inspectors, and our work was much facilitated by the co-operation of the Board and its staff. Every stress sheet and every calculation was investigated and checked, and every detail was checked to the last rivet. Material was inspected at the mills, in the shop, and in the field. Workmanship was most carefully inspected both in the shop and in the field, and all field engineering (lines, levels and measurements) was carefully checked. It was very reassuring to have this supervision and to feel that it was practically impossible for an error to escape unnoticed.

CONTENTS

PHOENIX BRIDGE DESIG



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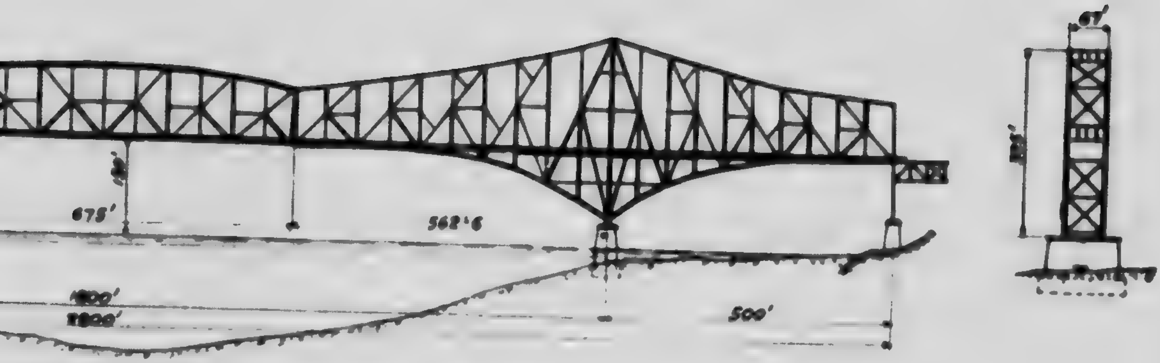
OUTLINE PHOEN



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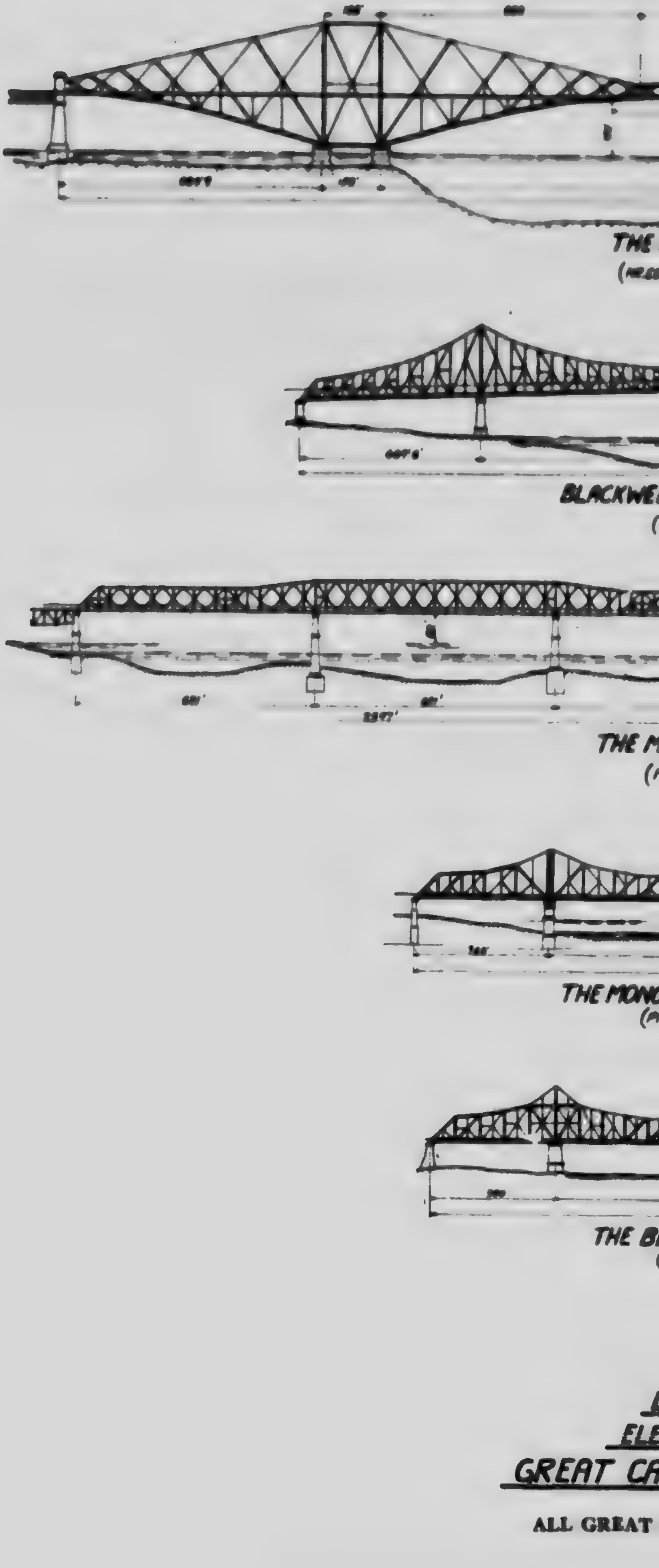
PHOENIX AND OFFICIAL DESIGN

PLATE I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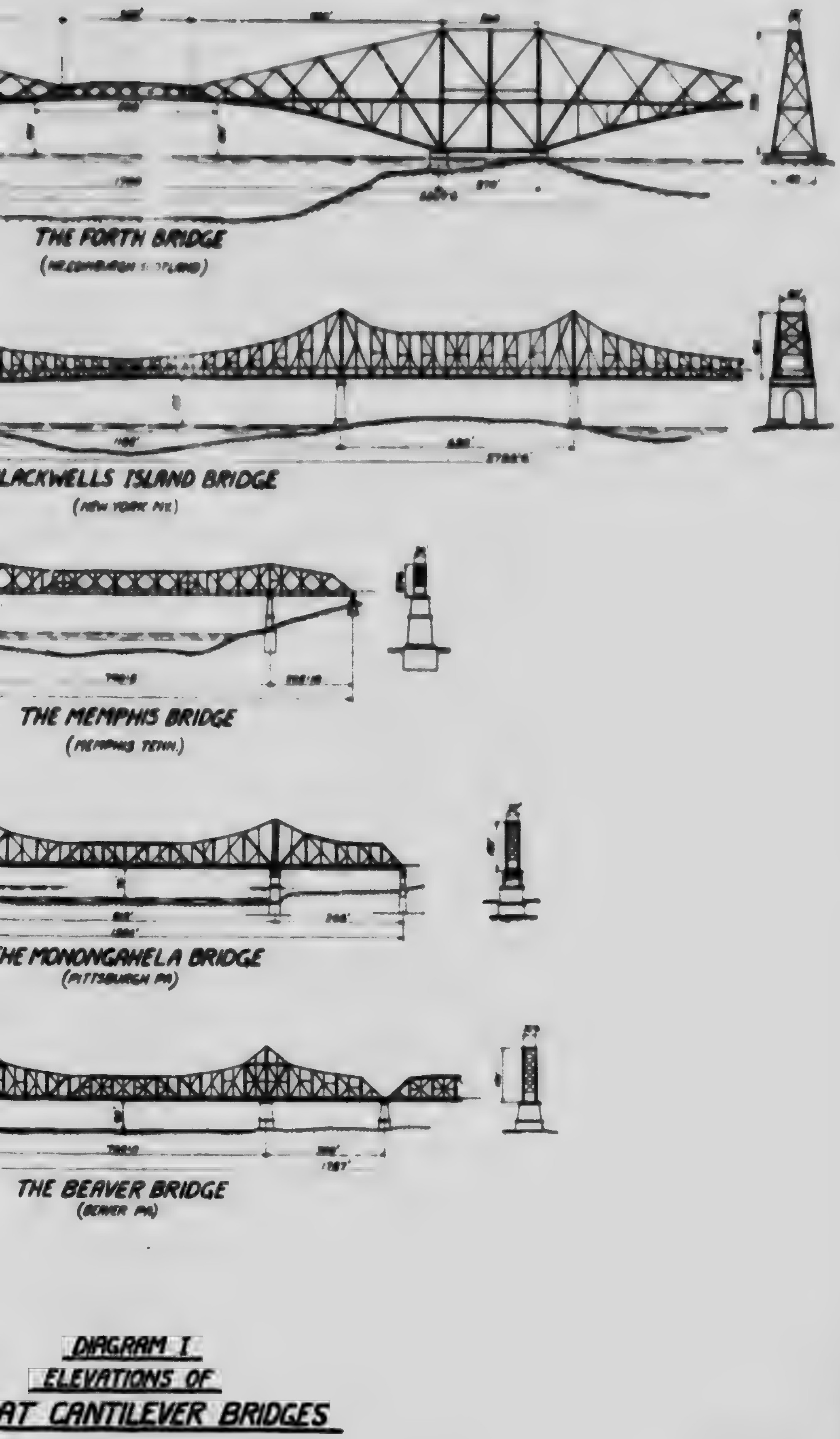
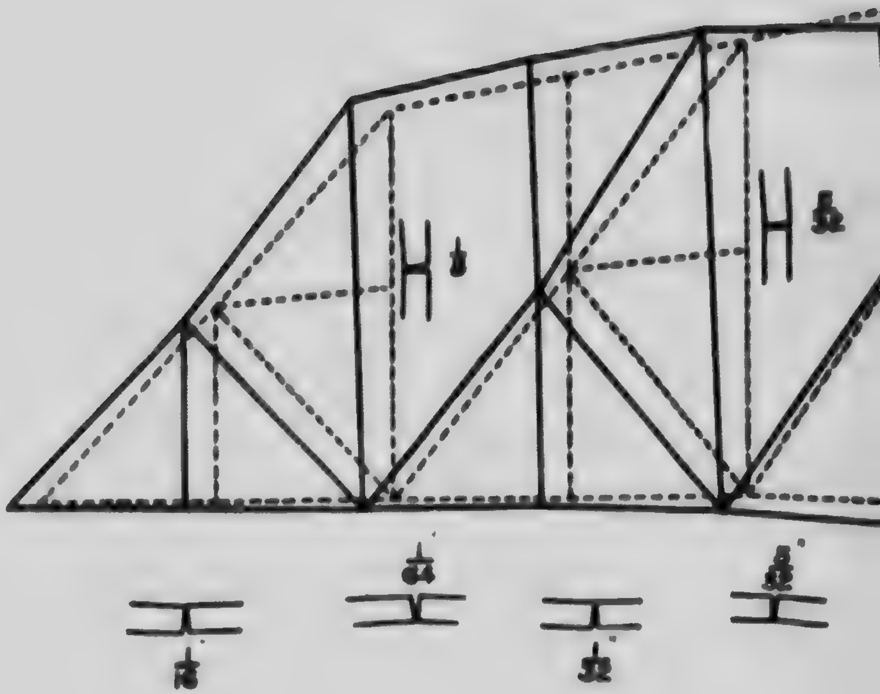


GREAT CANTILEVER BRIDGES PLATE II



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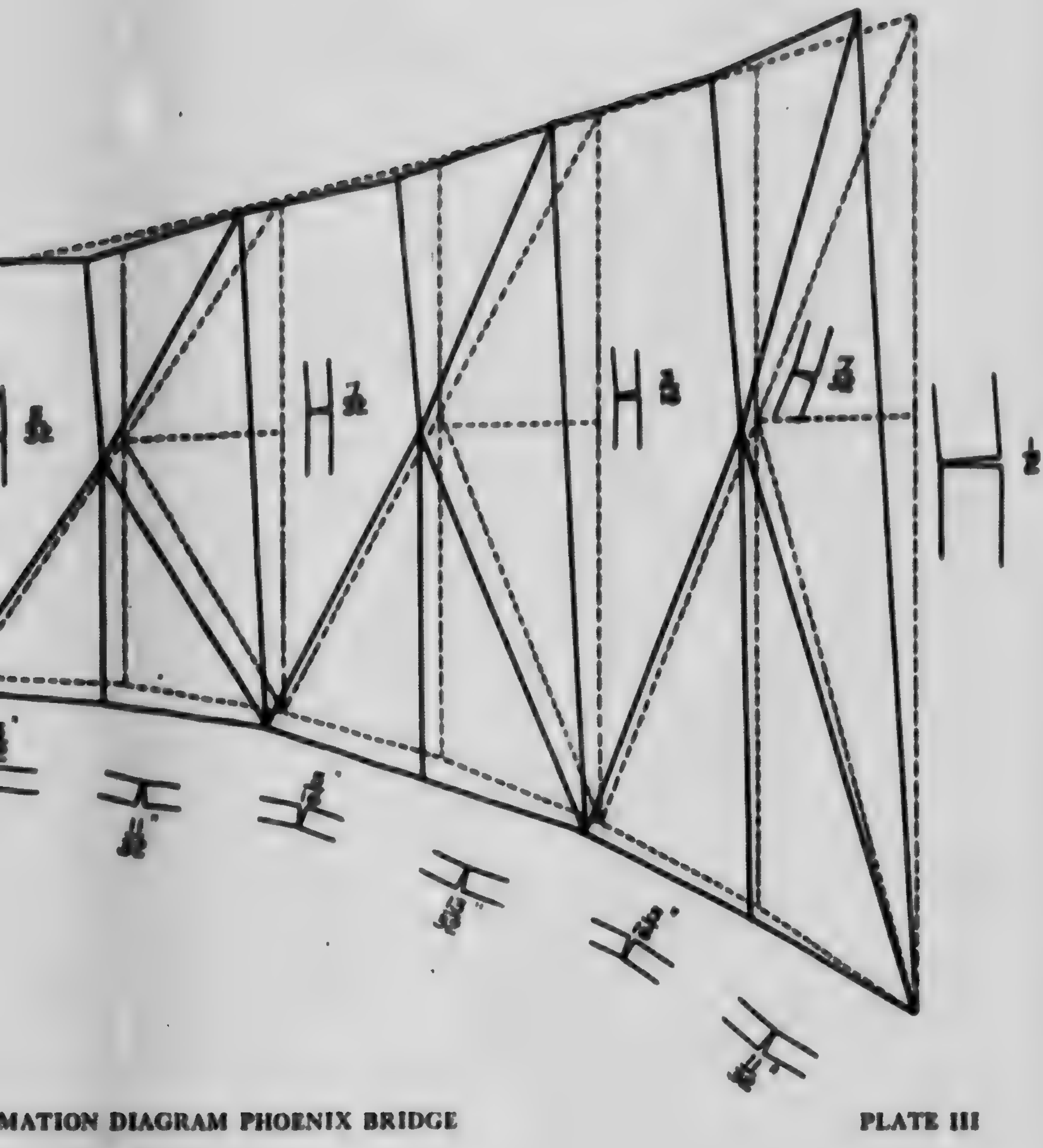
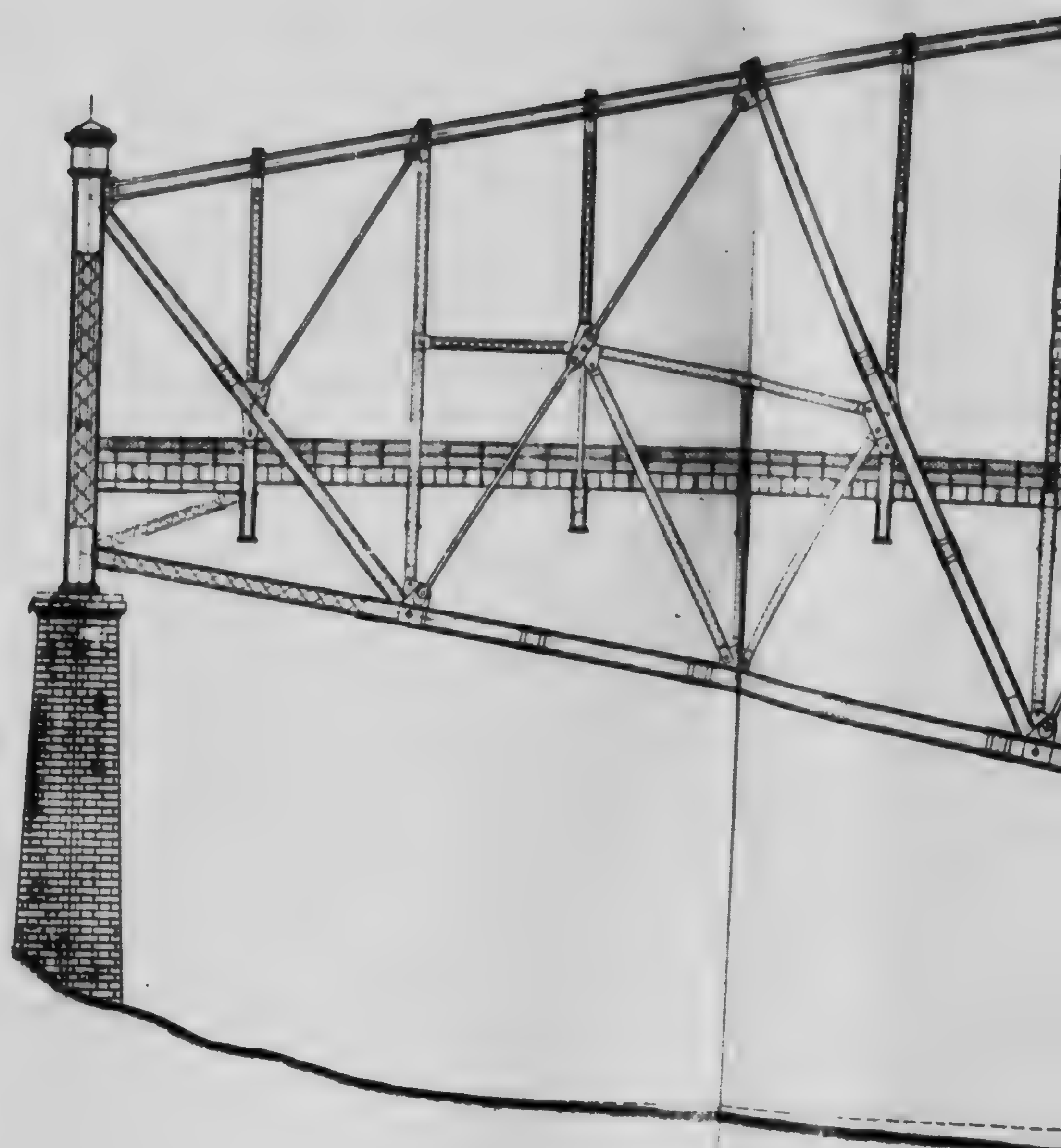
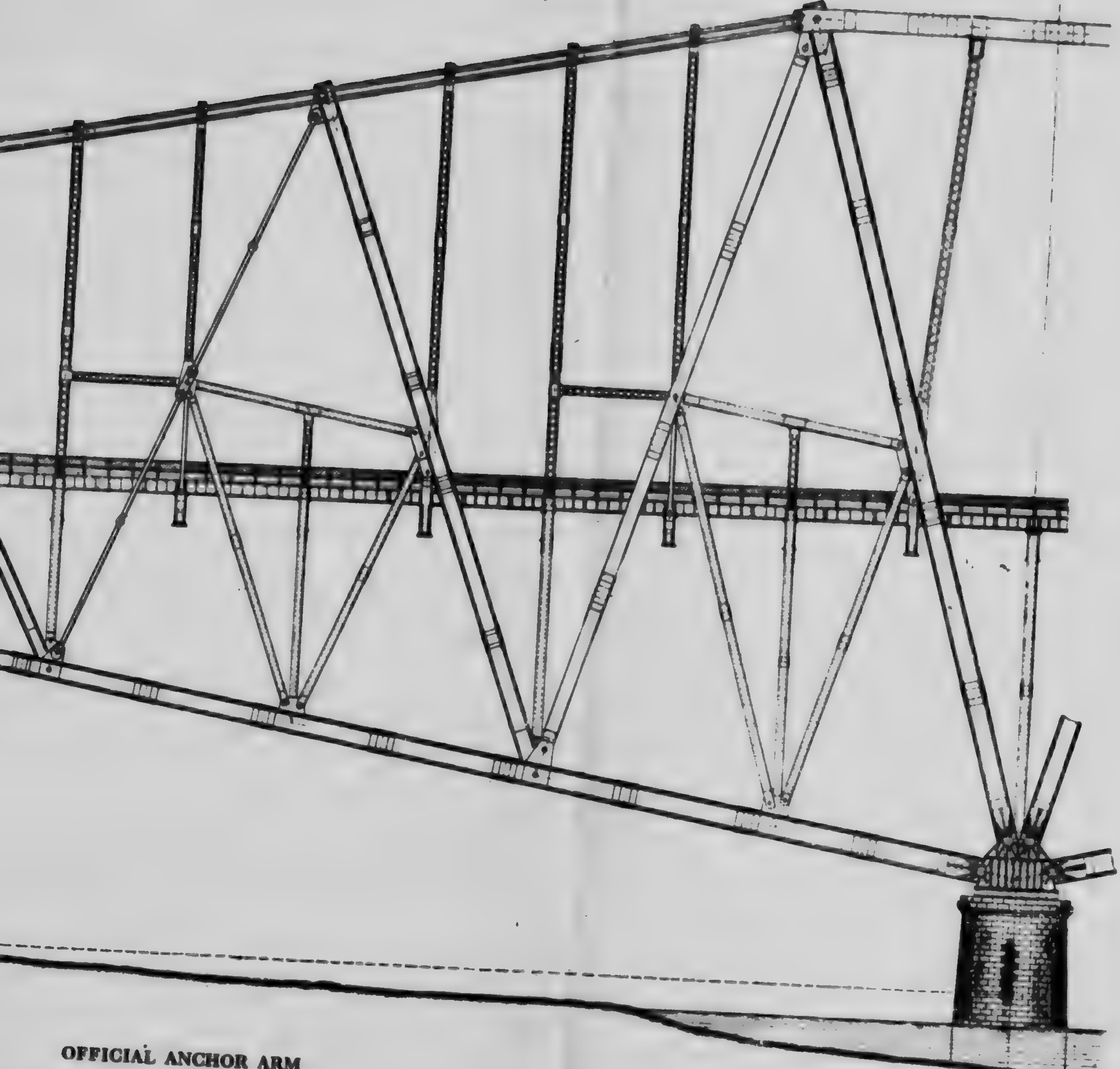


MATION DIAGRAM PHOENIX BRIDGE

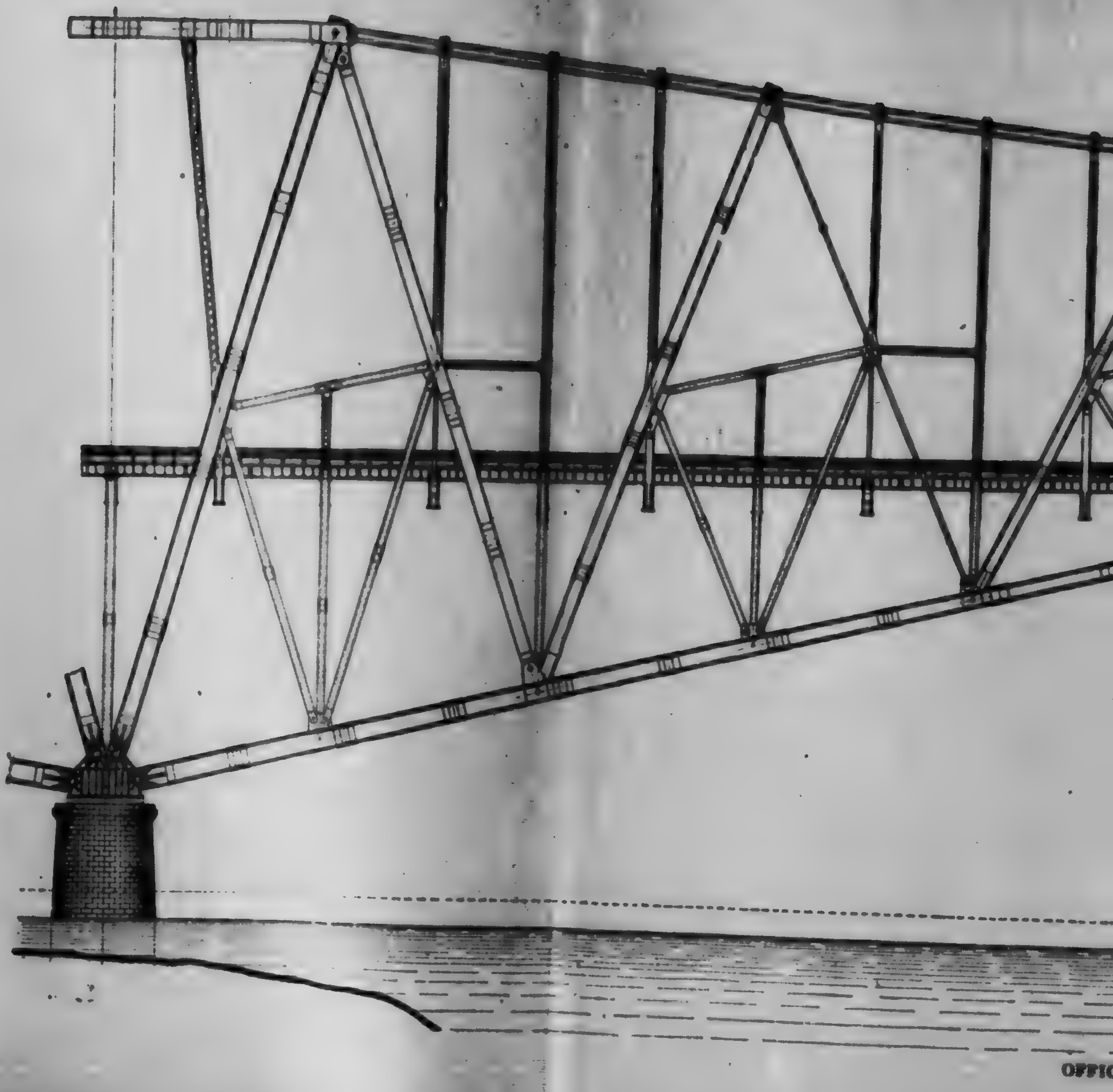
PLATE III





OFFICIAL ANCHOR ARM

PLATE IV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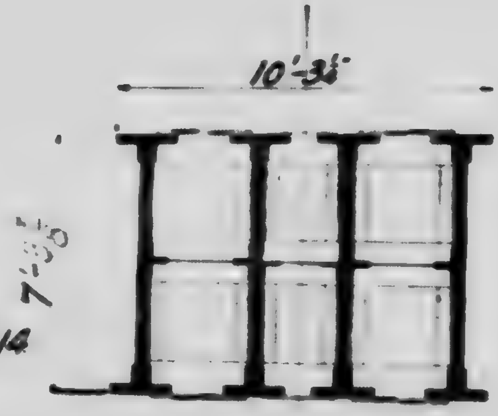
OFFICIAL CANTILEVER ARM

PLATE V

Quebec

New Design

Lower Chord. AL13 AL14



Area. 1941 □"

Forth

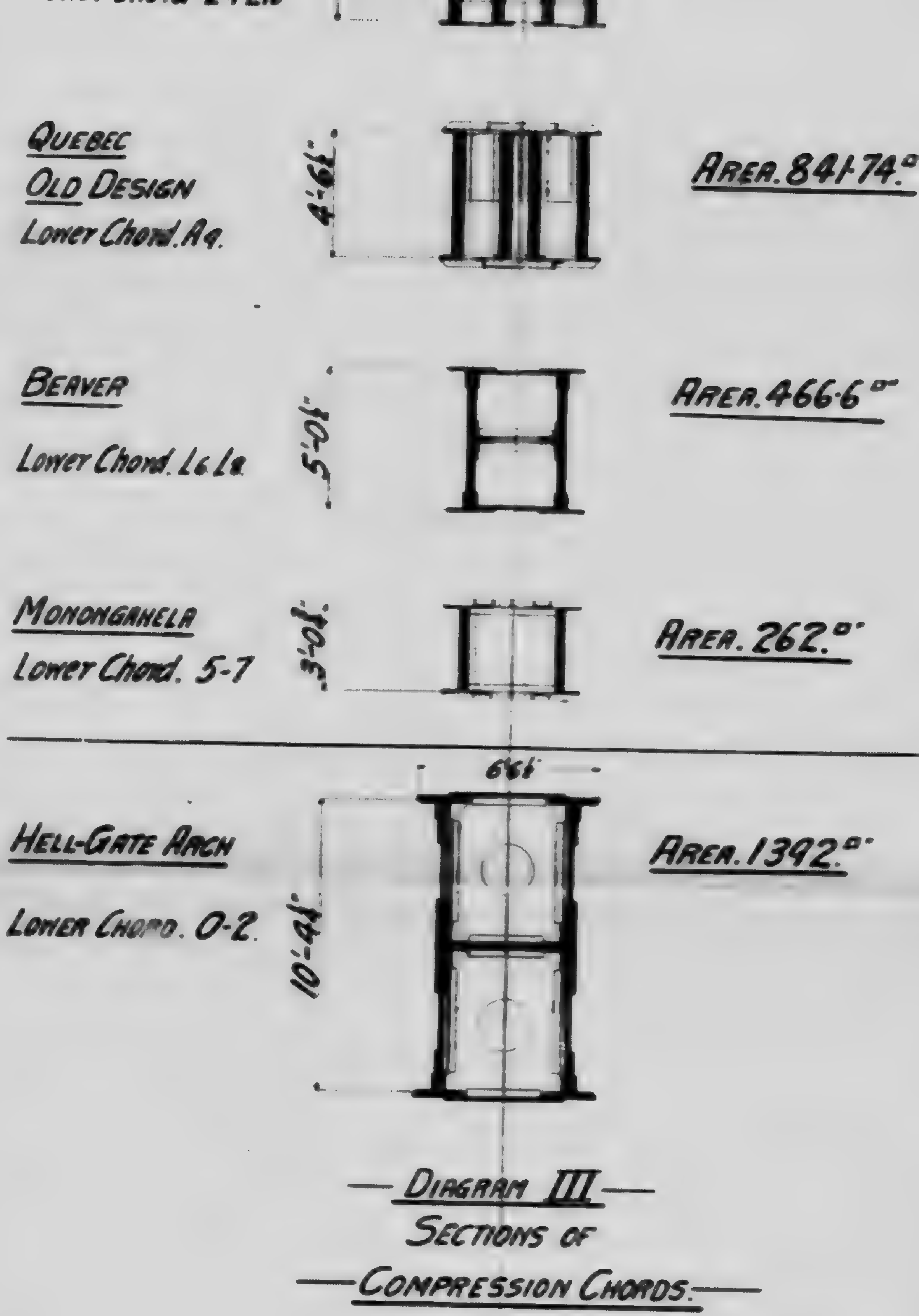
Tube Section

Area. 800 □"

Blackwells Island

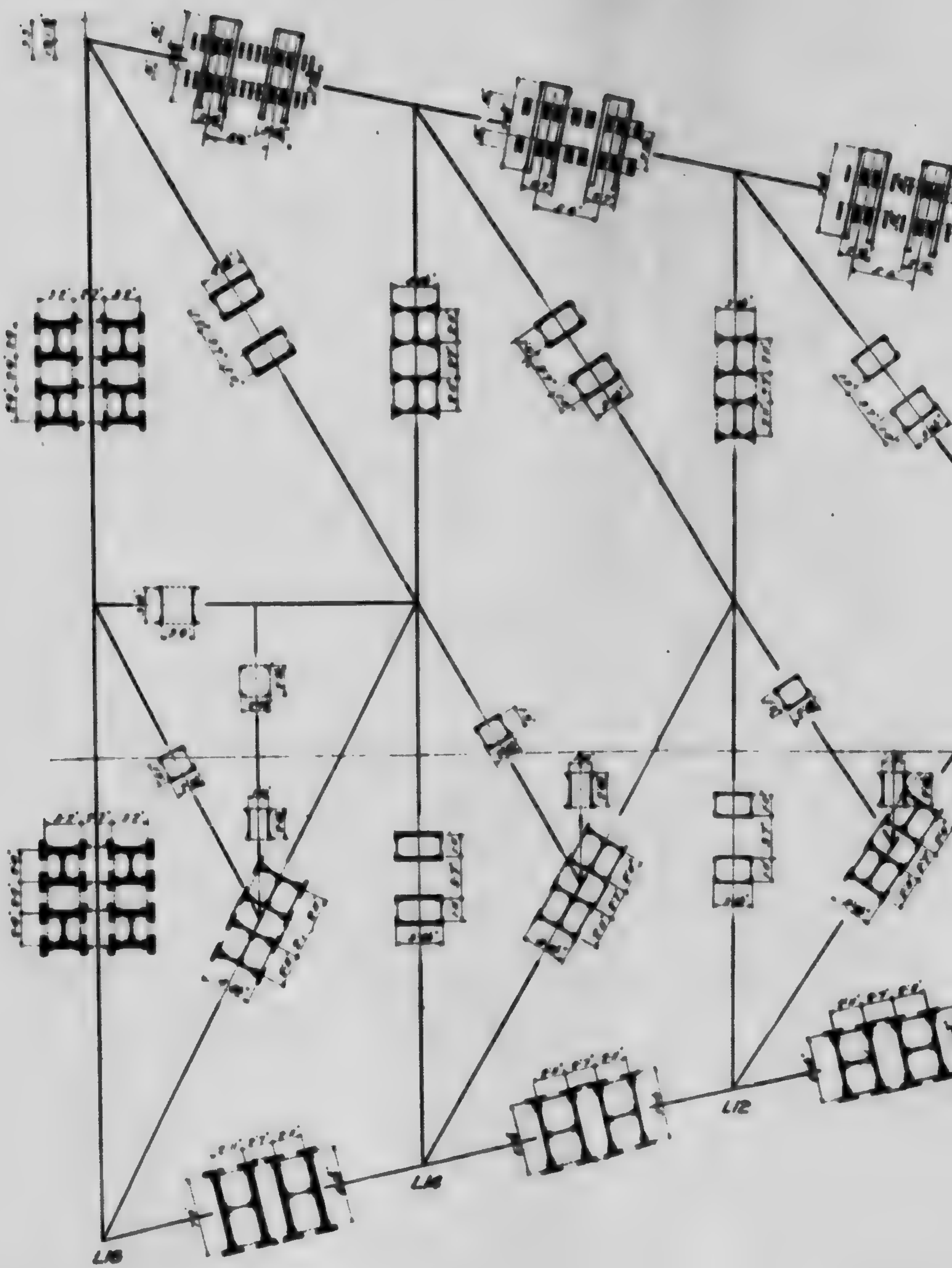
Lower Chord L9 L10

Area. 853·16 □"



LARGE CHORD SECTIONS

PLATE VI



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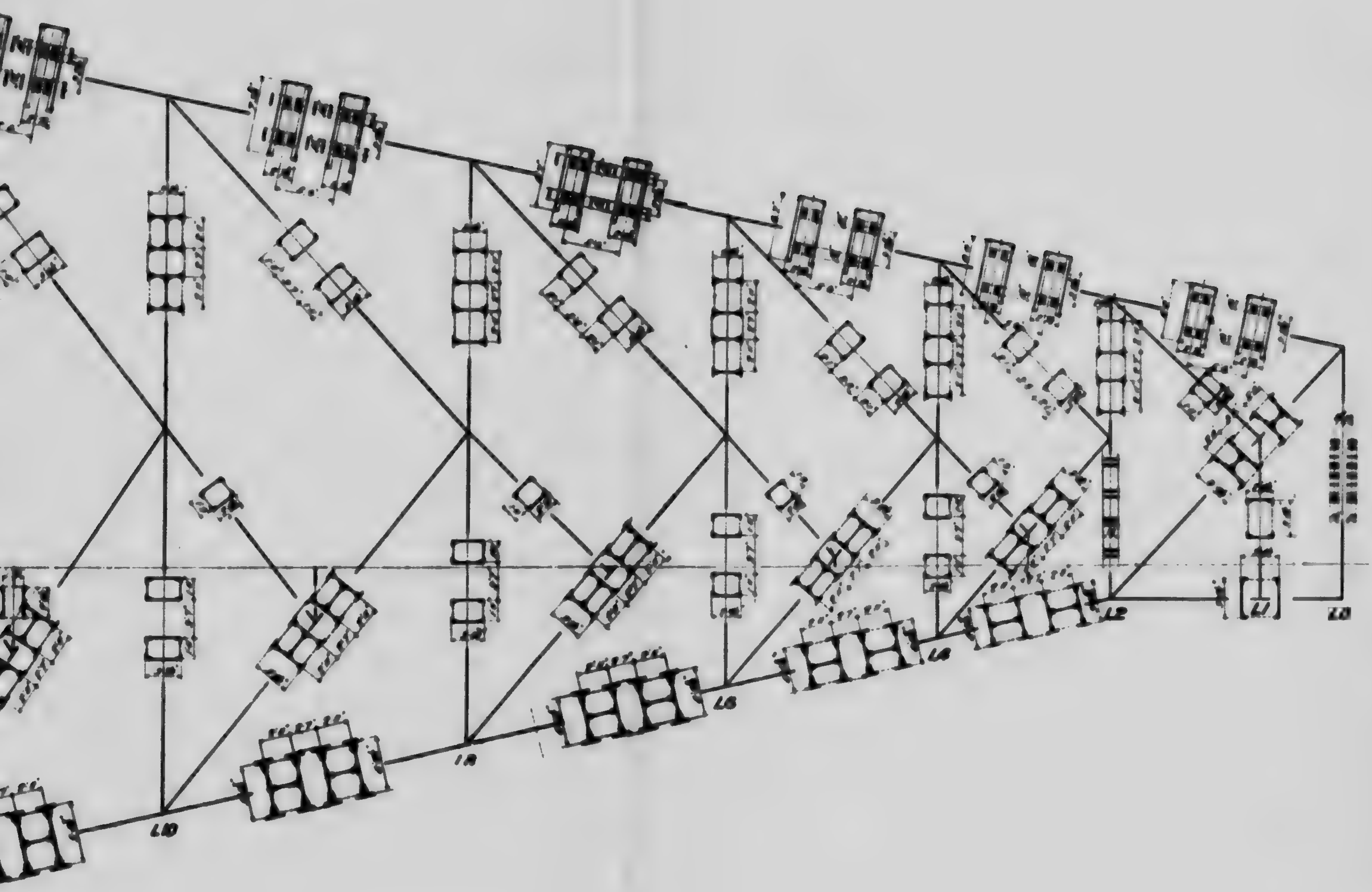
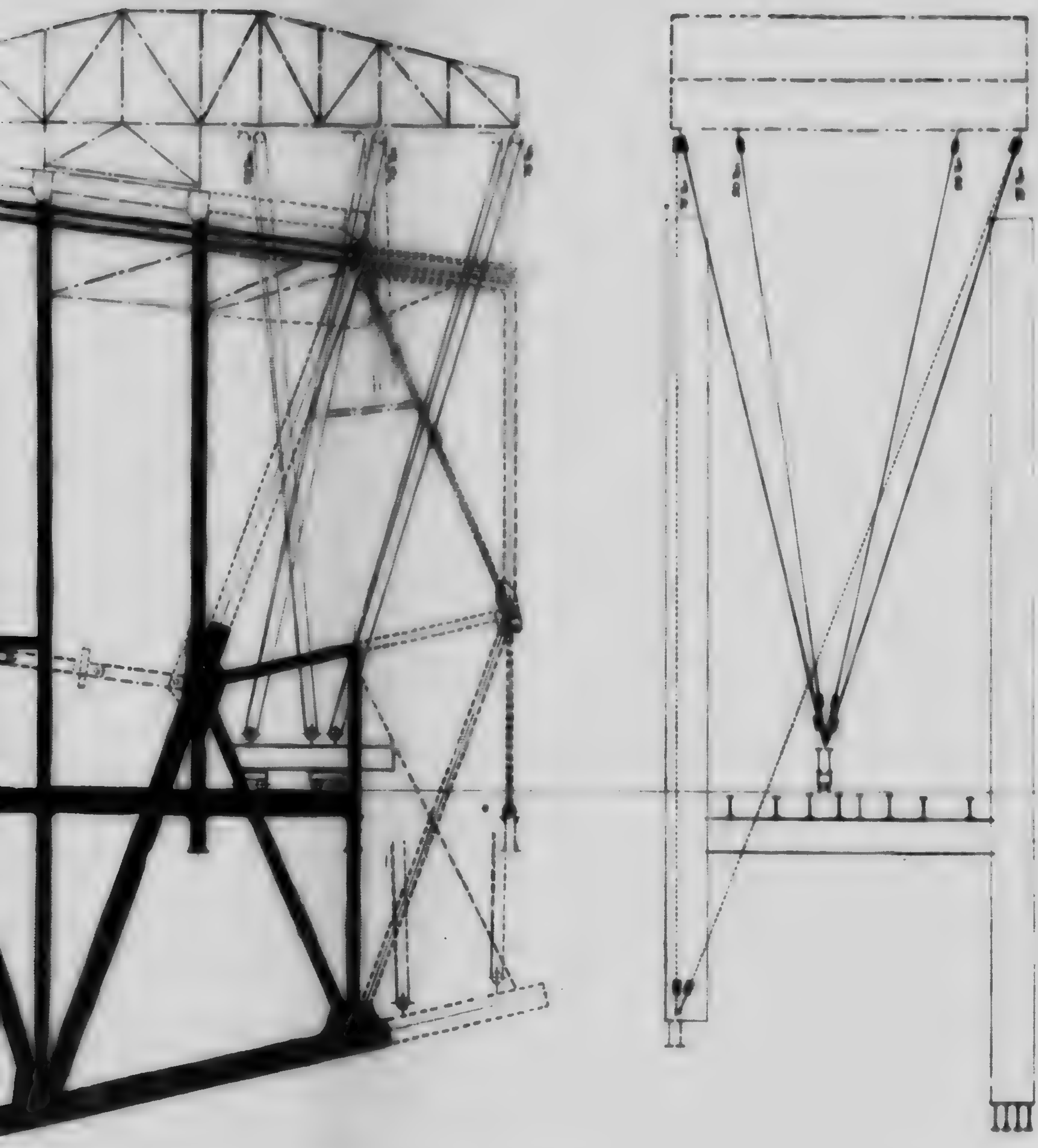


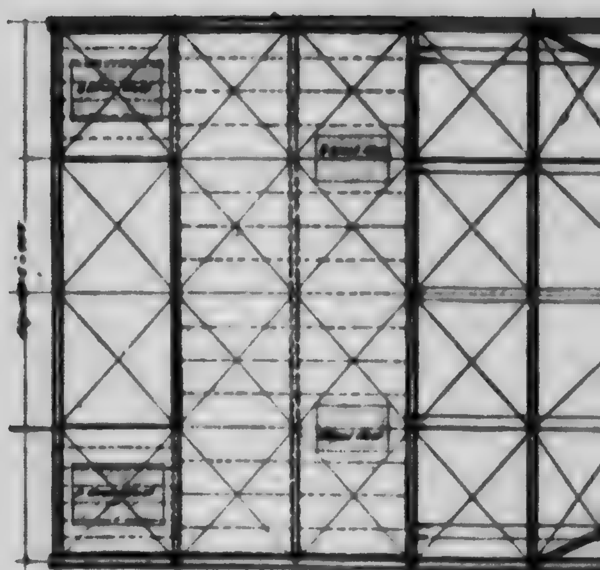
CROSS SECTIONS IMPORTANT MEMBERS

PLATE VII



GENERAL CHARACTER ERECTION (OFFICIAL)

PLATE VIII



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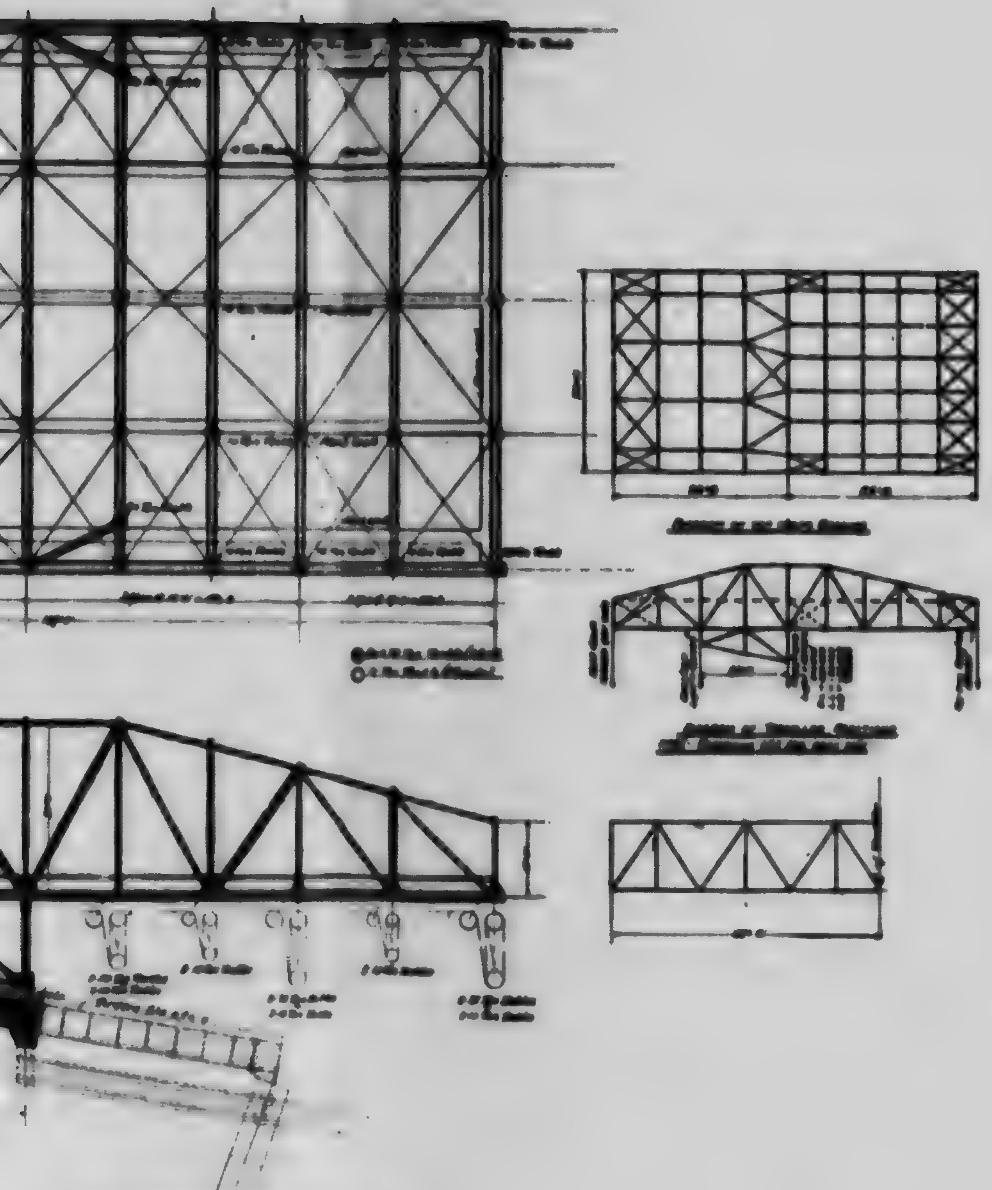
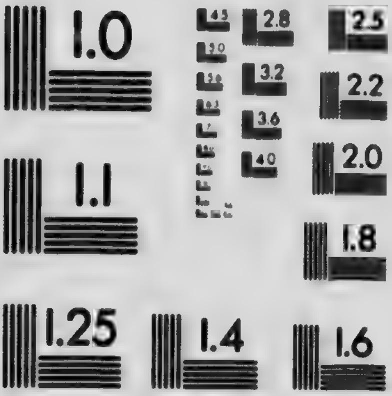


TOP CHORD TRAVELLER

PLATE IX

MICROCOPY RESOLUTION TEST CHART

(ANSI and ISO TEST CHART No. 2)



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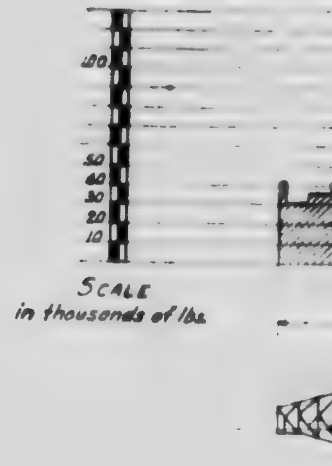
(716) 288 - 5989 - Fax



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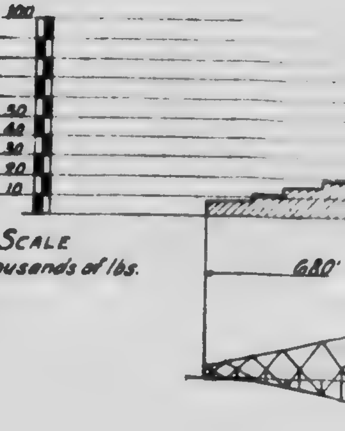

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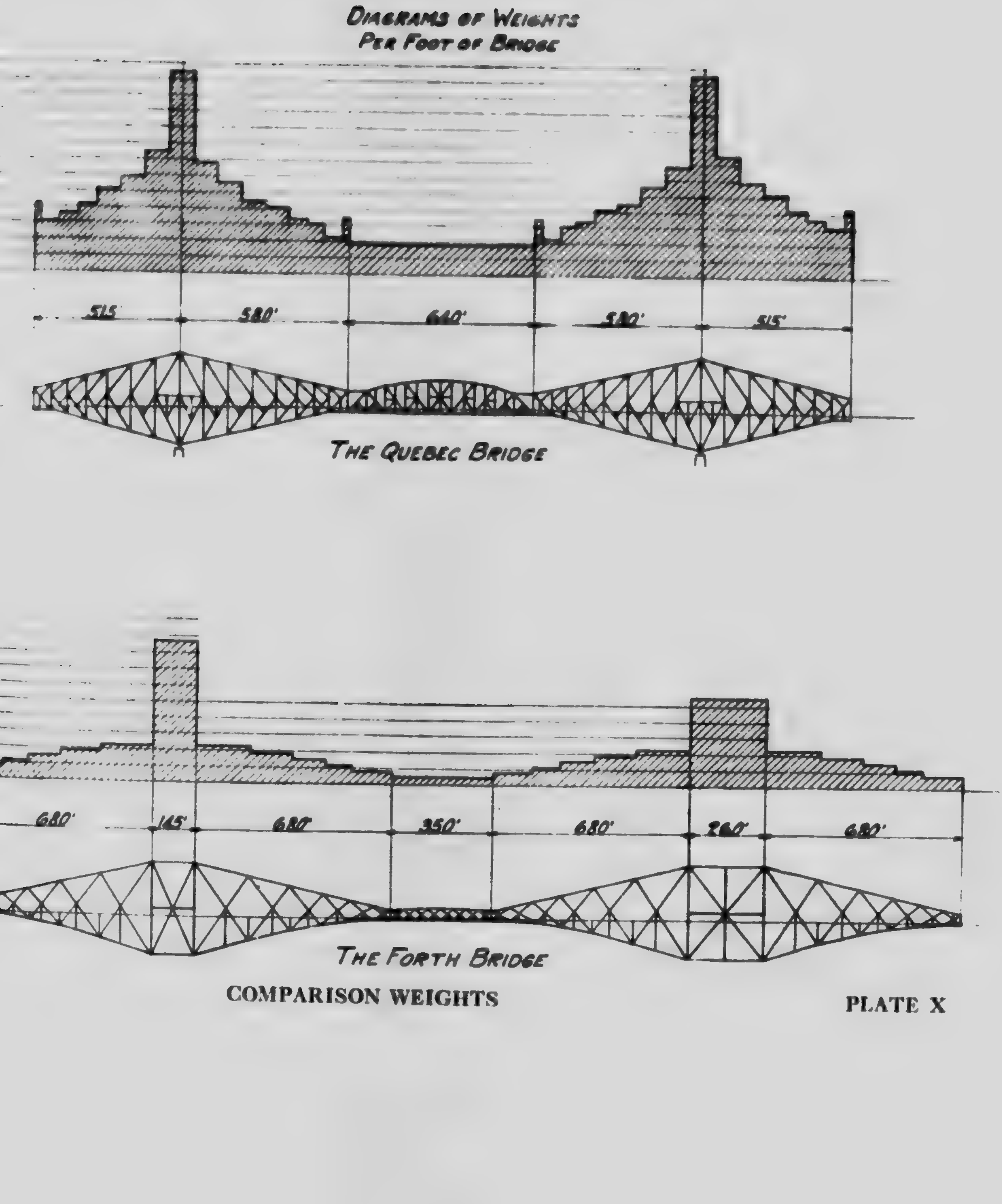
in thousands of lbs.
680'



COMPARISON WEIGHTS

PLATE X

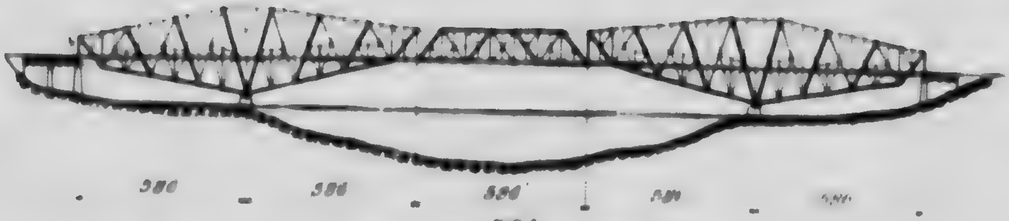


FIG 1
BOARD OF ENGINEERS
QUEBEC BRIDGE
DESIGN I

FIG 2
BOARD OF ENGINEERS
QUEBEC BRIDGE
DESIGN V

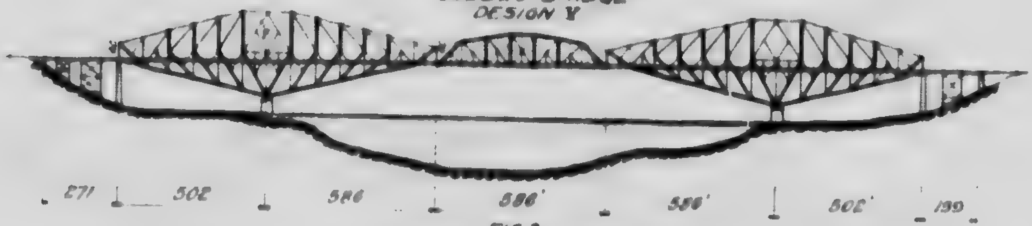


FIG 3
S^T LAWRENCE BRIDGE C^O L^TD
QUEBEC BRIDGE
DESIGN A·B·X

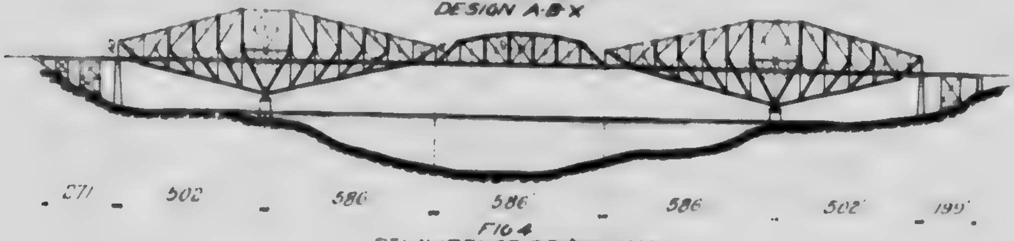


FIG 4
S^T LAWRENCE BRIDGE C^O L^TD
QUEBEC BRIDGE
DESIGN C·Y

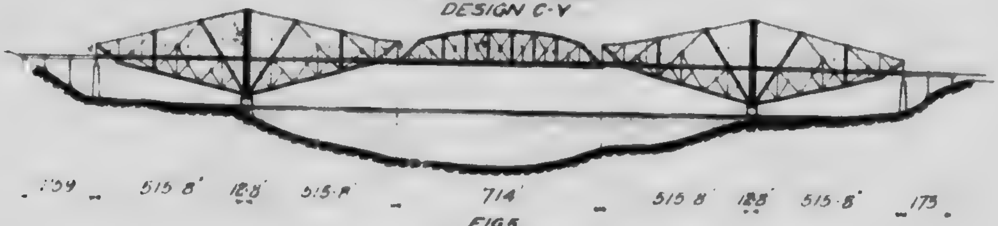


FIG 5
S^T LAWRENCE BRIDGE C^O L^TD
QUEBEC BRIDGE
DESIGN M·N

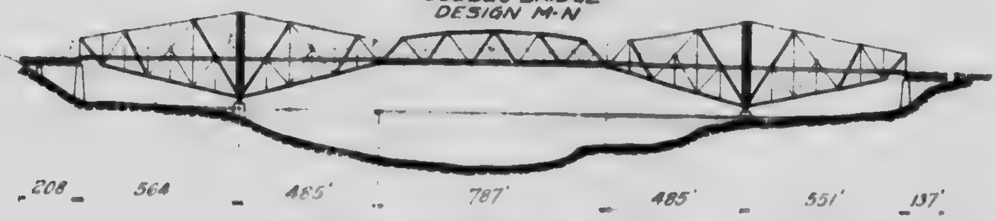
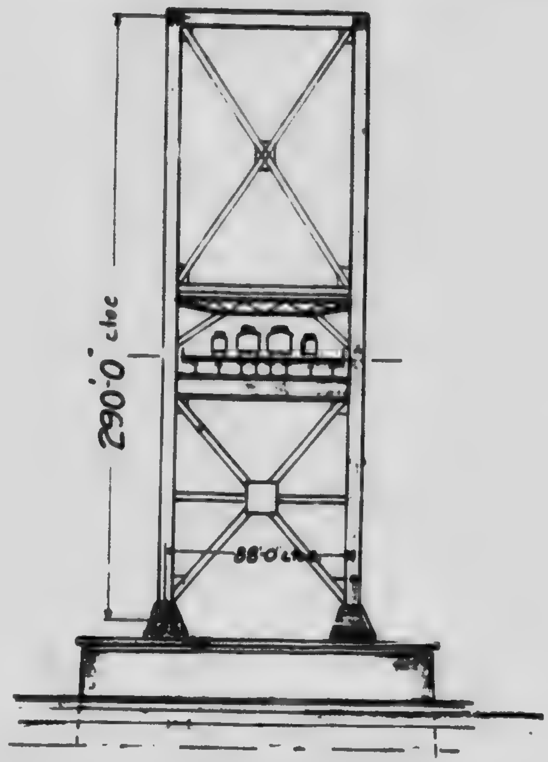


FIG 6
MASCHINENFABRIK AUGSBURG-NÜRNBERG A.G.
QUEBEC BRIDGE.

FIG 7
PENNSYLVANIA STEEL C^O
SUSPENSION BRIDGE
DESIGN FOR QUEBEC
BRIDGE

OUTLINE OF DESIGNS SUBMITTED

PLATE XI

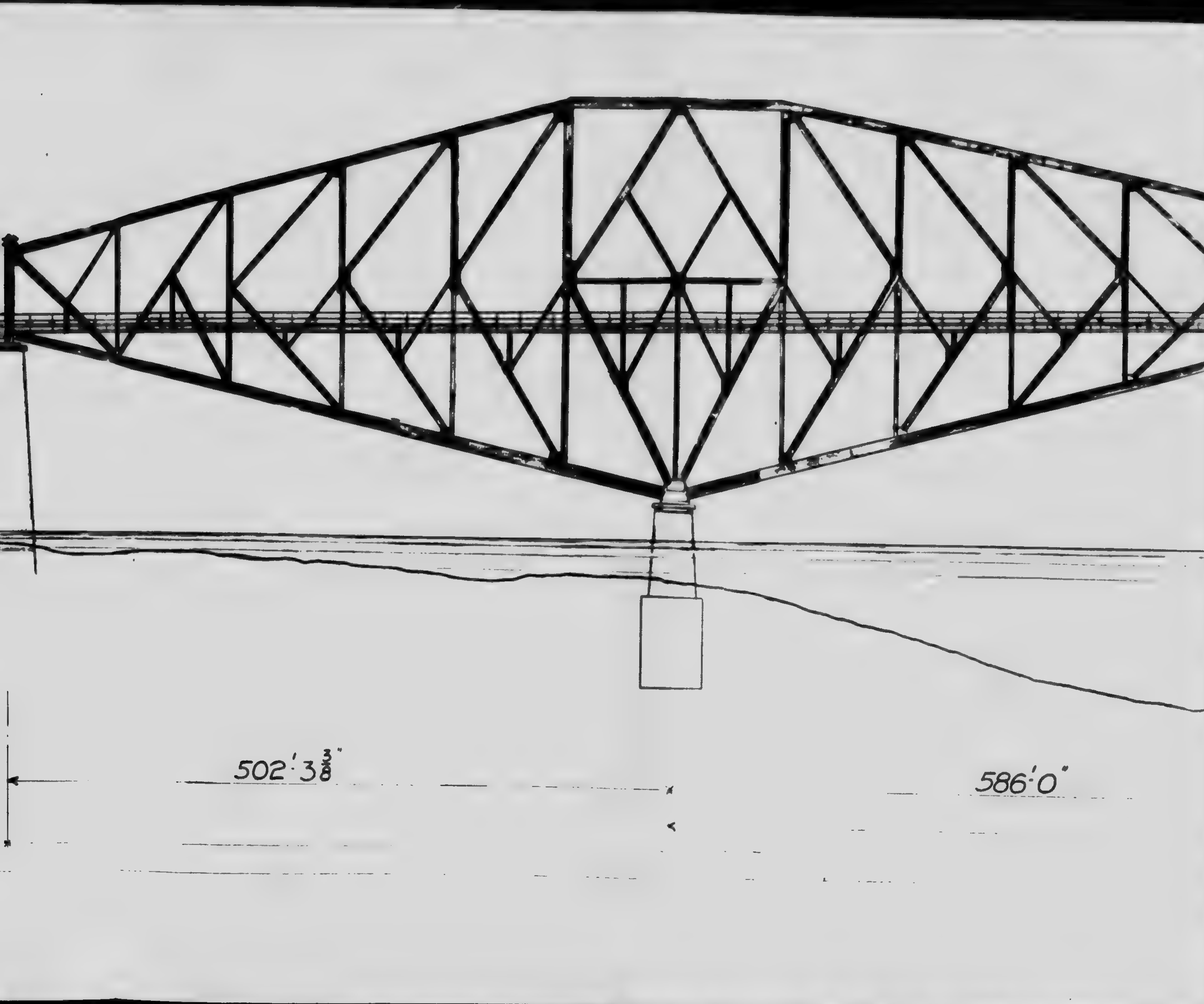


-DESIGN "B"-

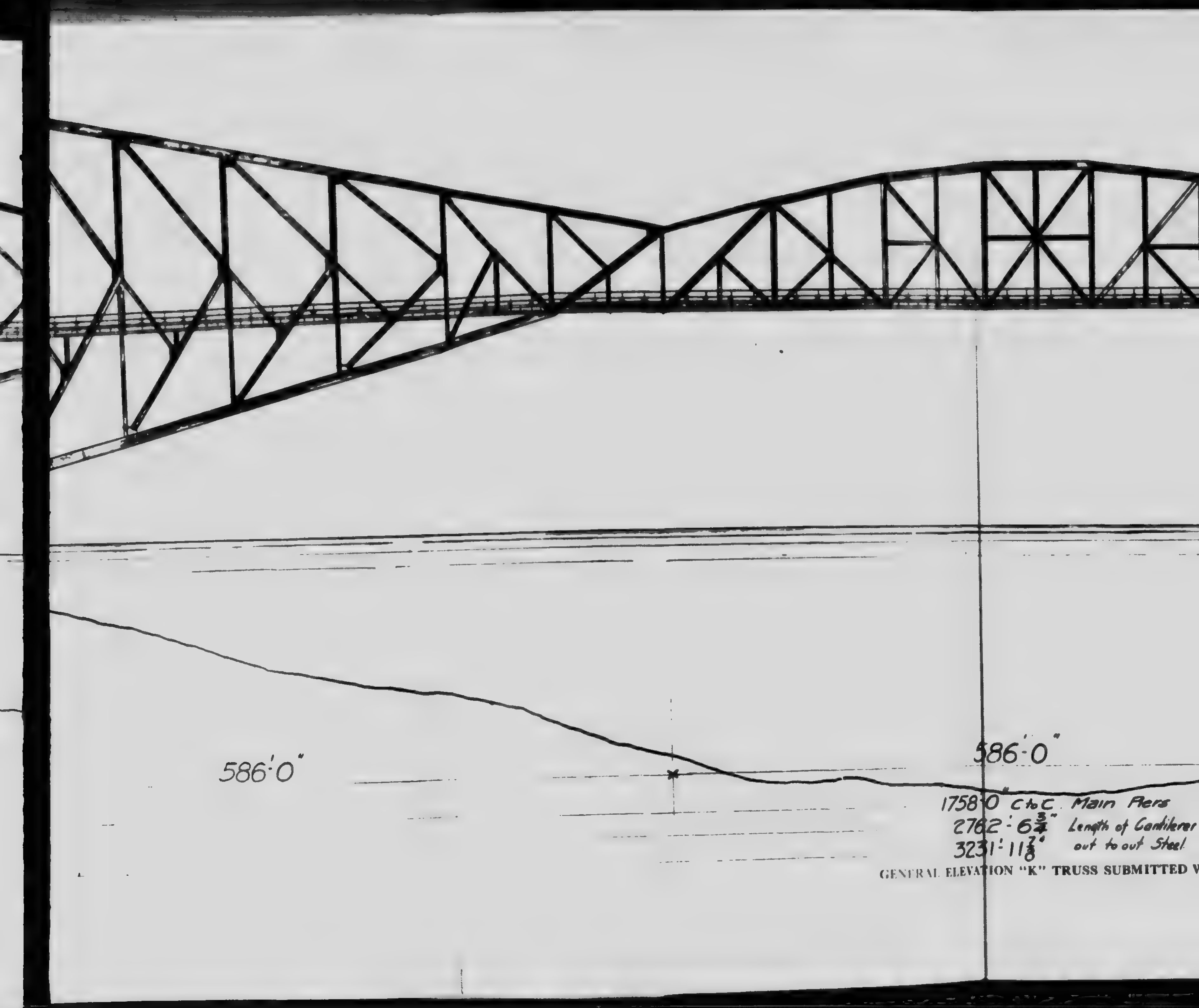
— Cross Section at Main Pier.—

-DESIGN "X"-

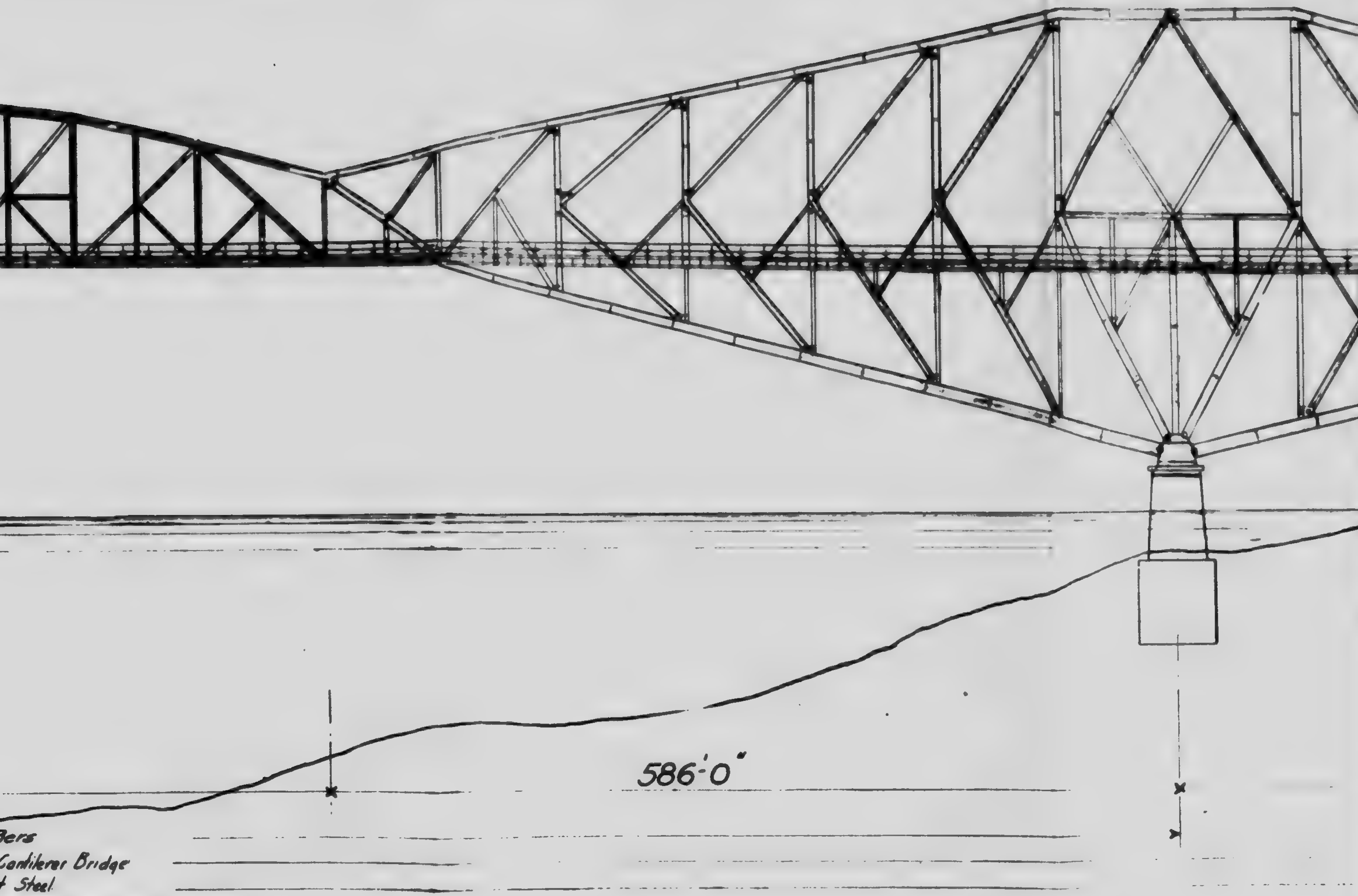
— Cross Section same as above with Electric Railway tracks and Highways omitted. —



502' 3 3/8"
586' 0"



GENERAL ELEVATION "K" TRUSS SUBMITTED W



HITTED WITH THE TENDERS

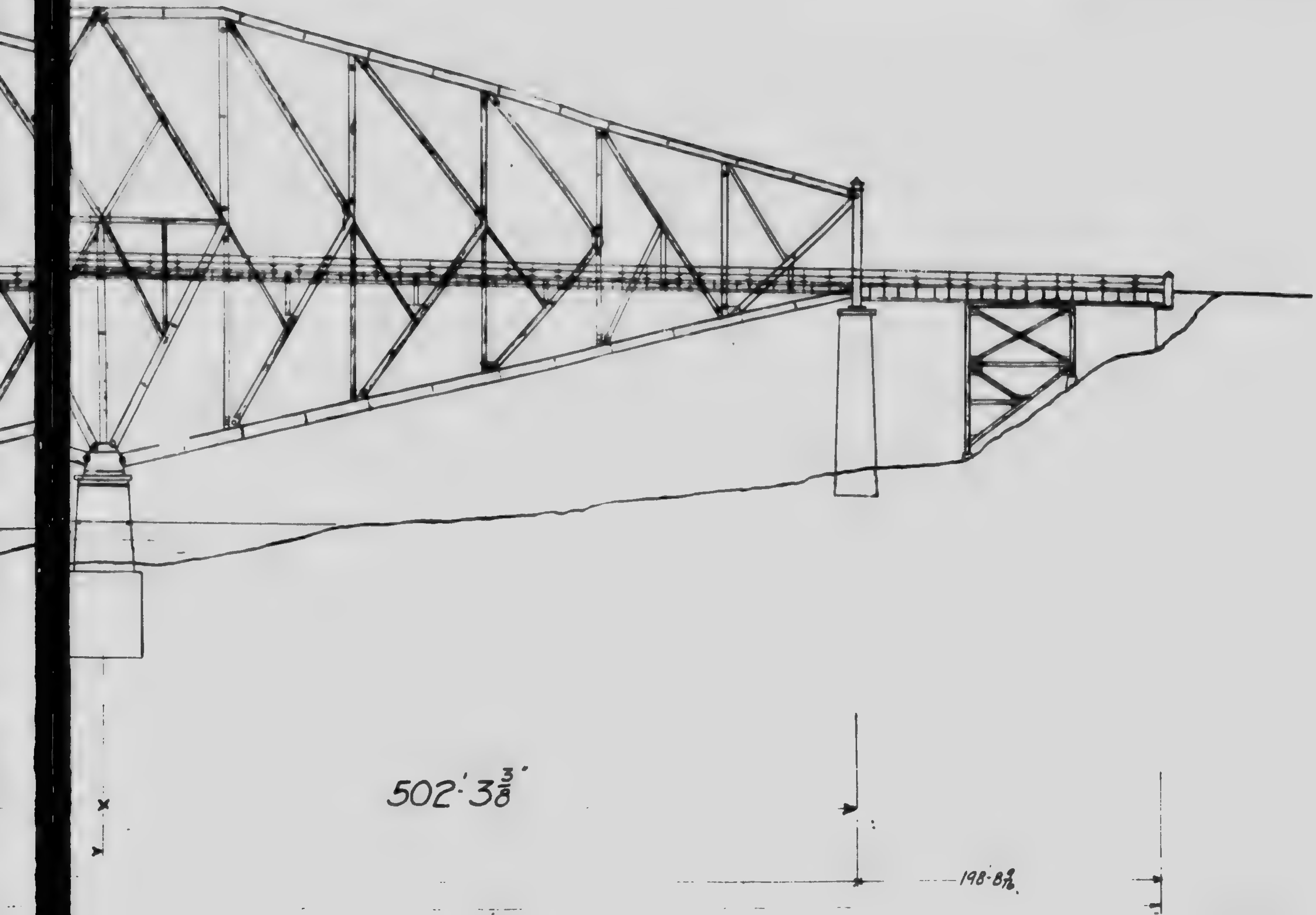
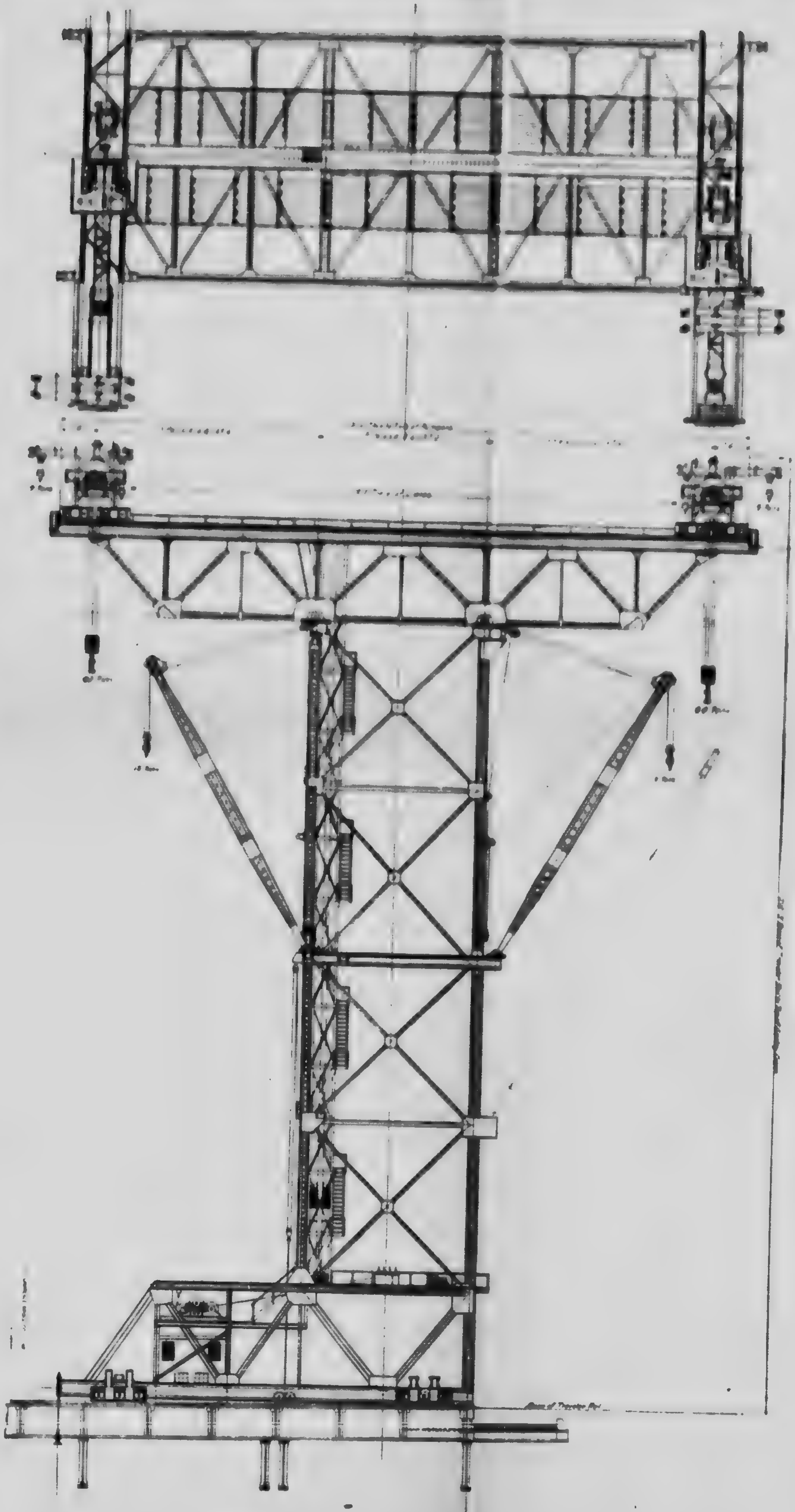
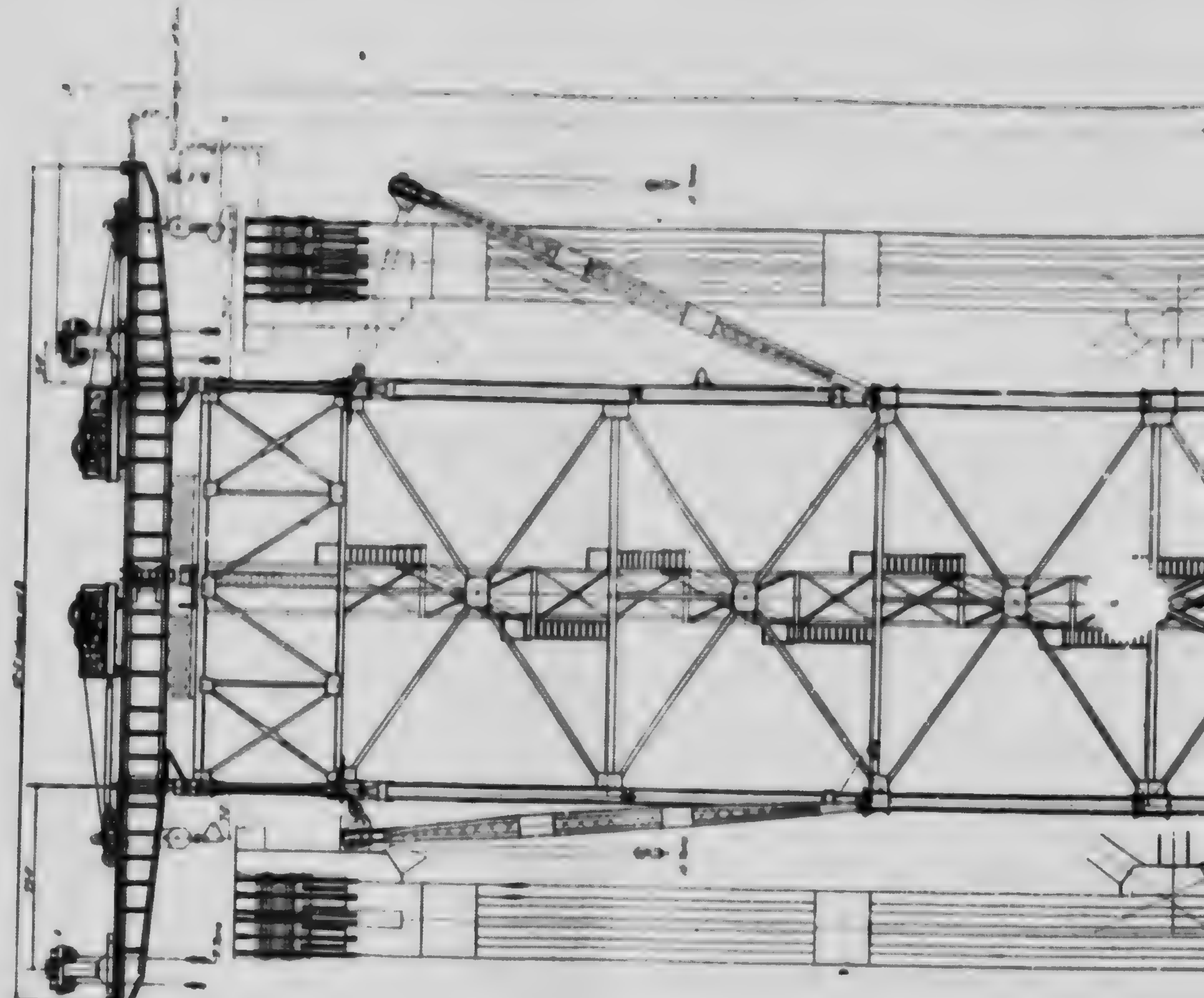


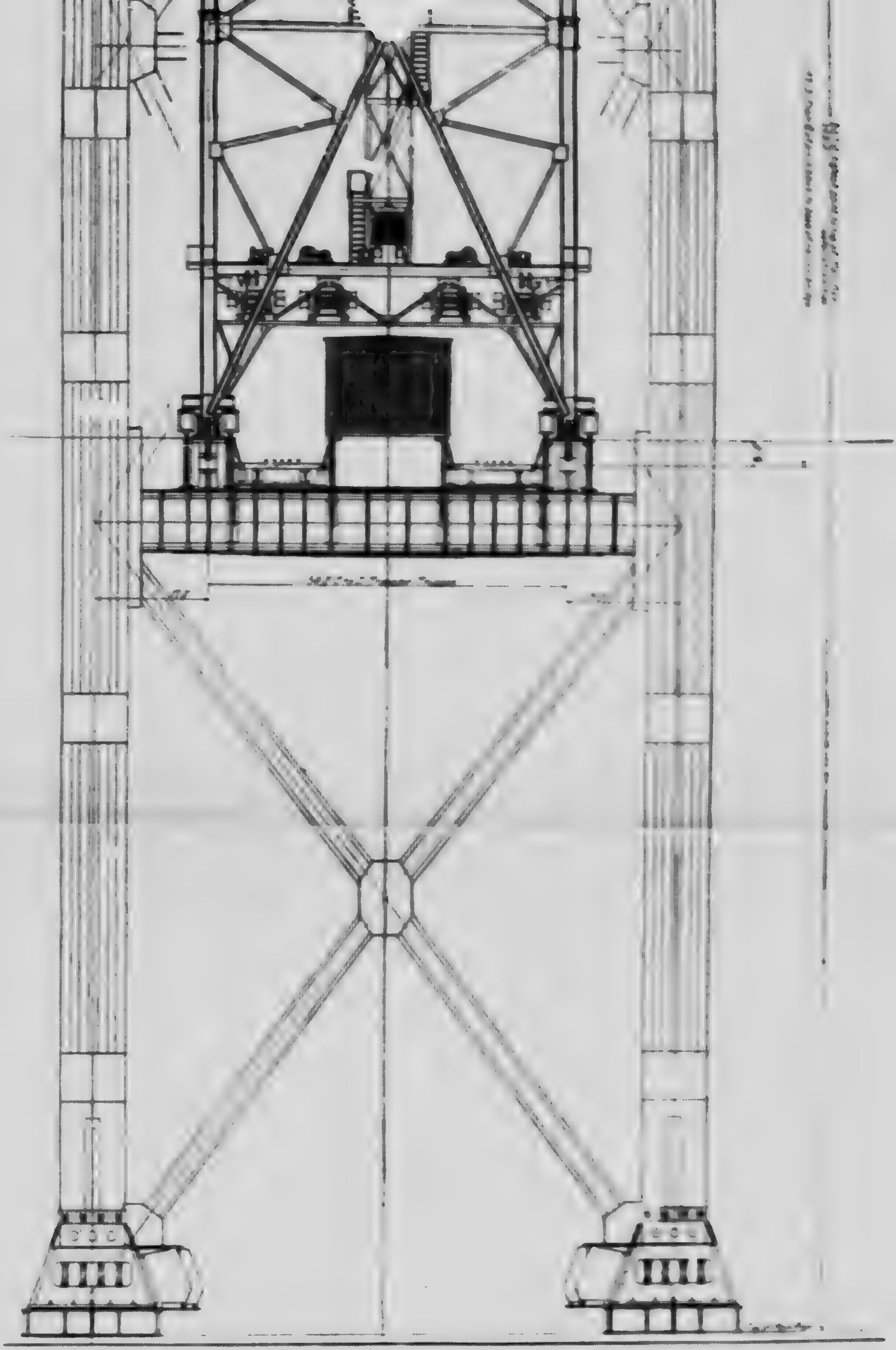
PLATE XII



TRAVELLER

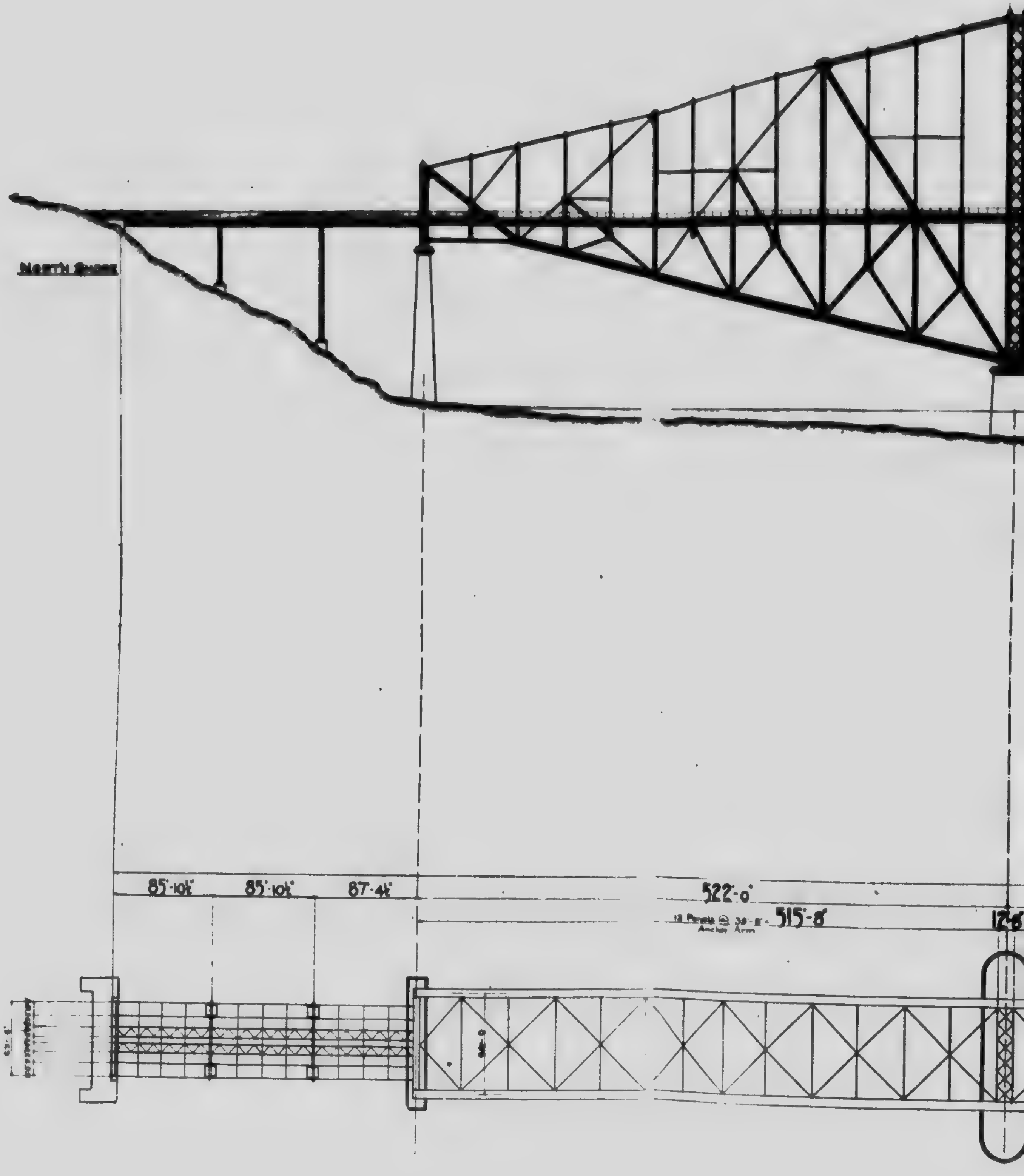
PLATE XIII



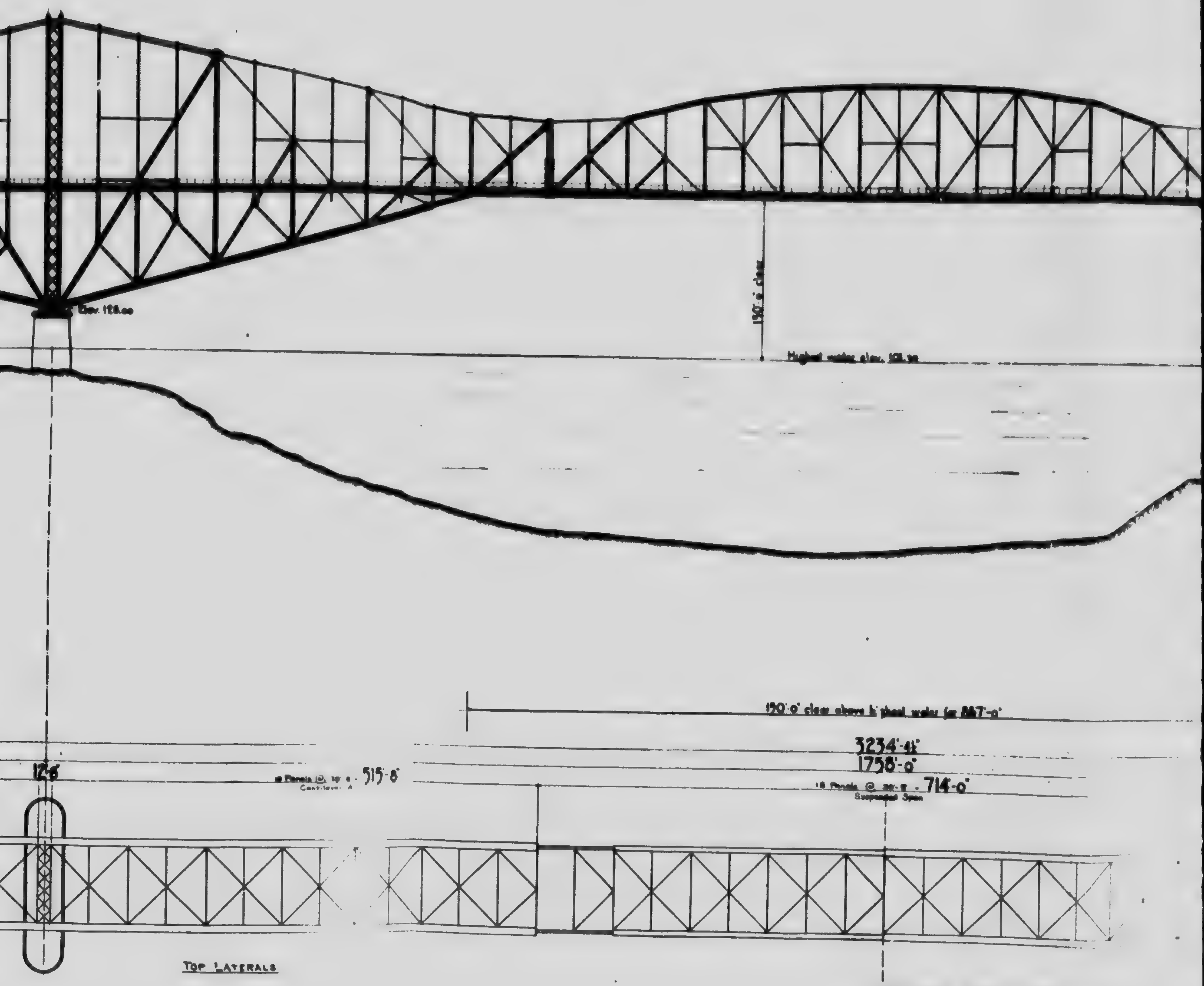


TRAVELLER

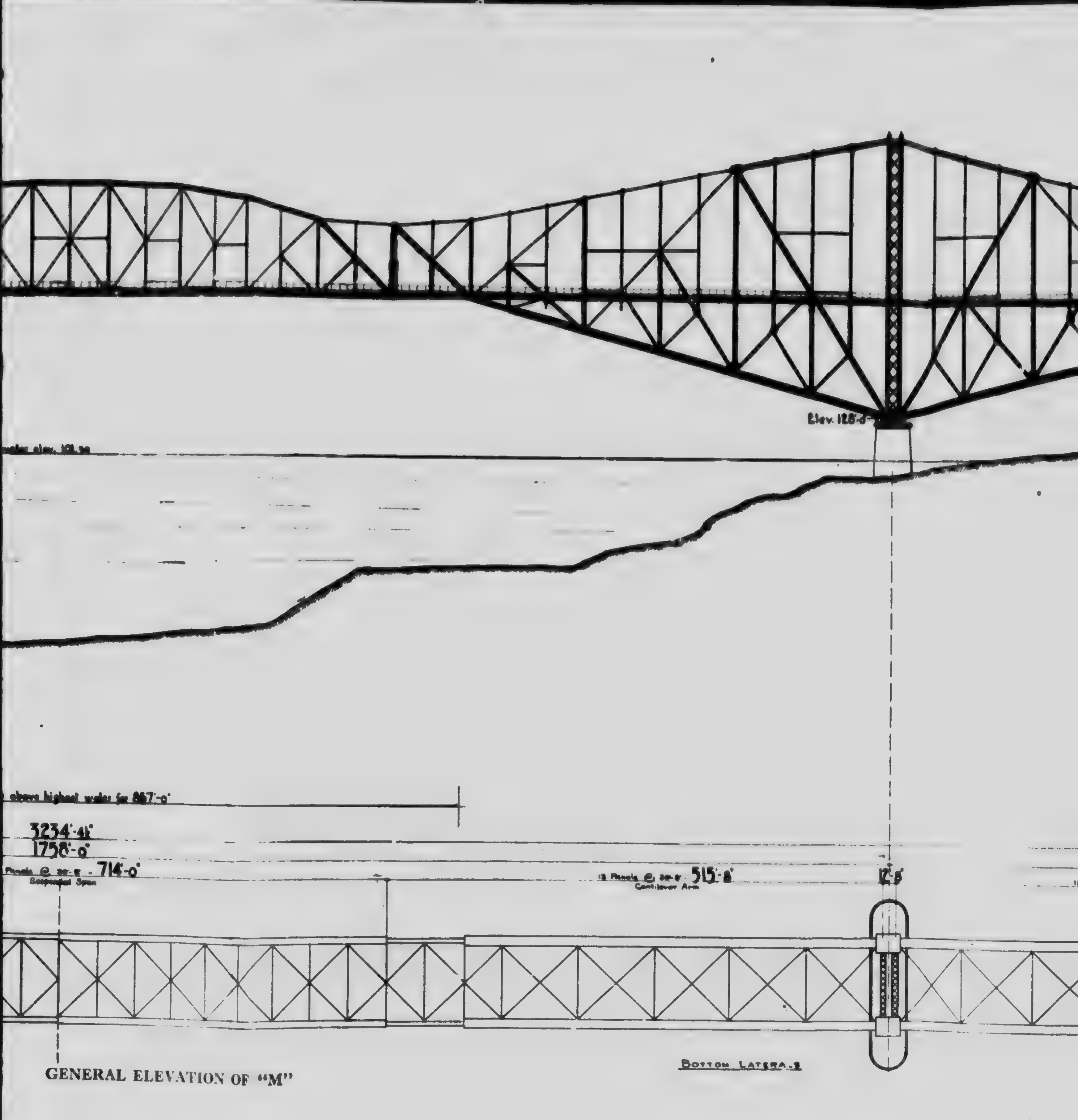
PLATE XIV



NORTH SHORE
85'-10½"
85'-10½"
87'-4½"
522'-0"
515'-8"
Anchor Arm
12'-6"



GENERAL ELEVATION OF "M"



GENERAL ELEVATION OF "M"

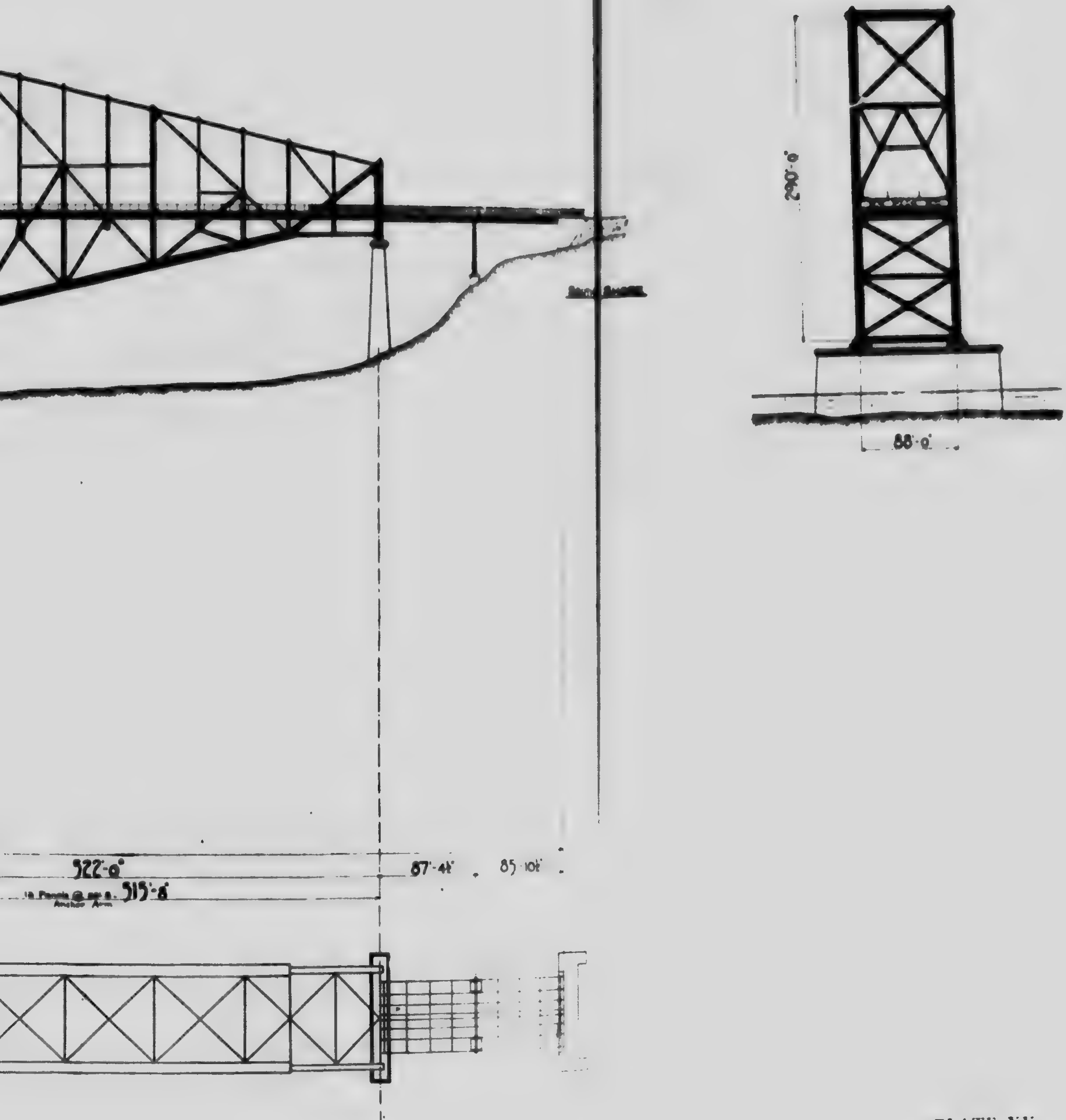
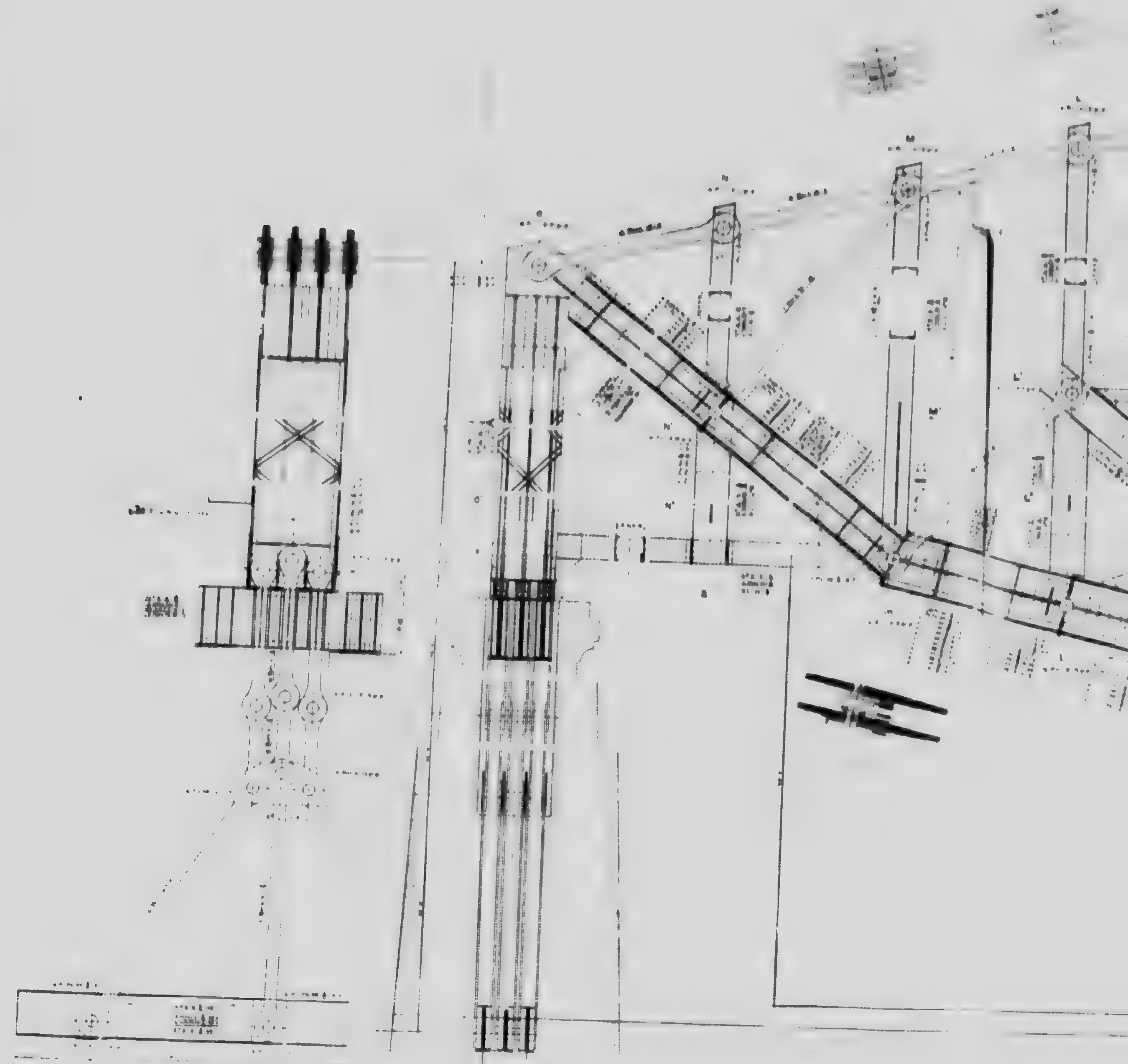
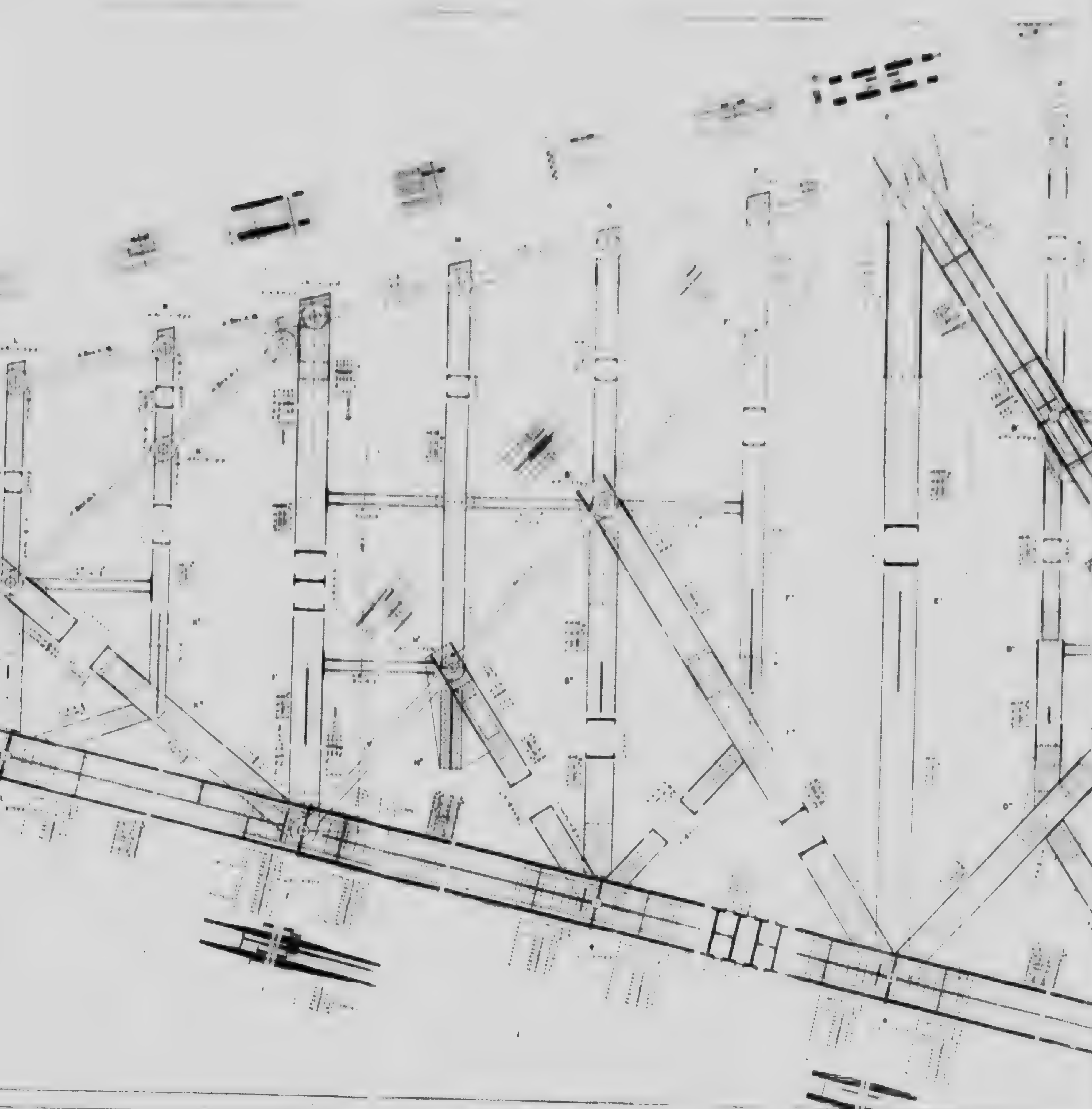
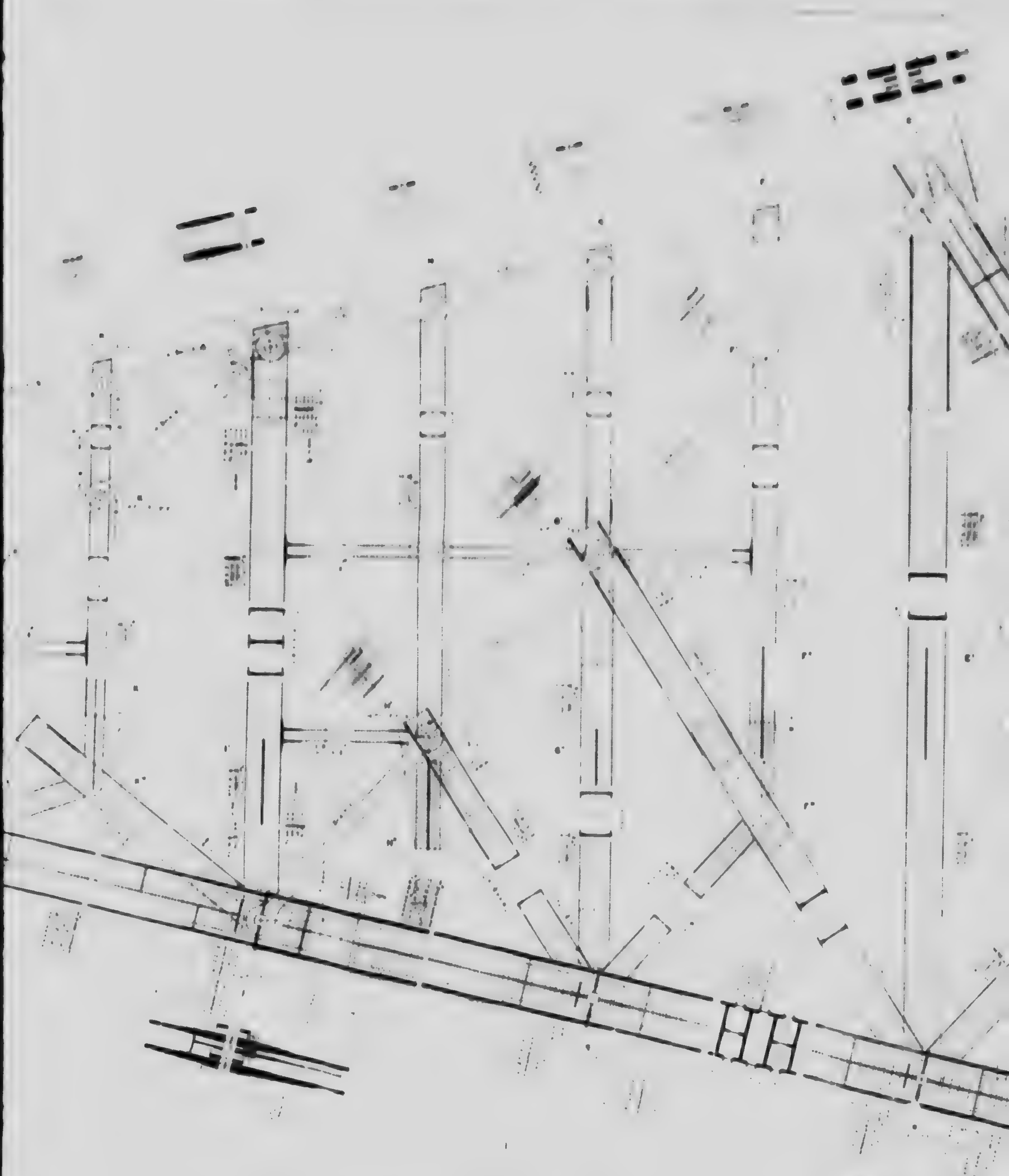


PLATE XV





PORTION OF "M", LARGER SCALE



POR'ION OF "M", LARGER SCALE

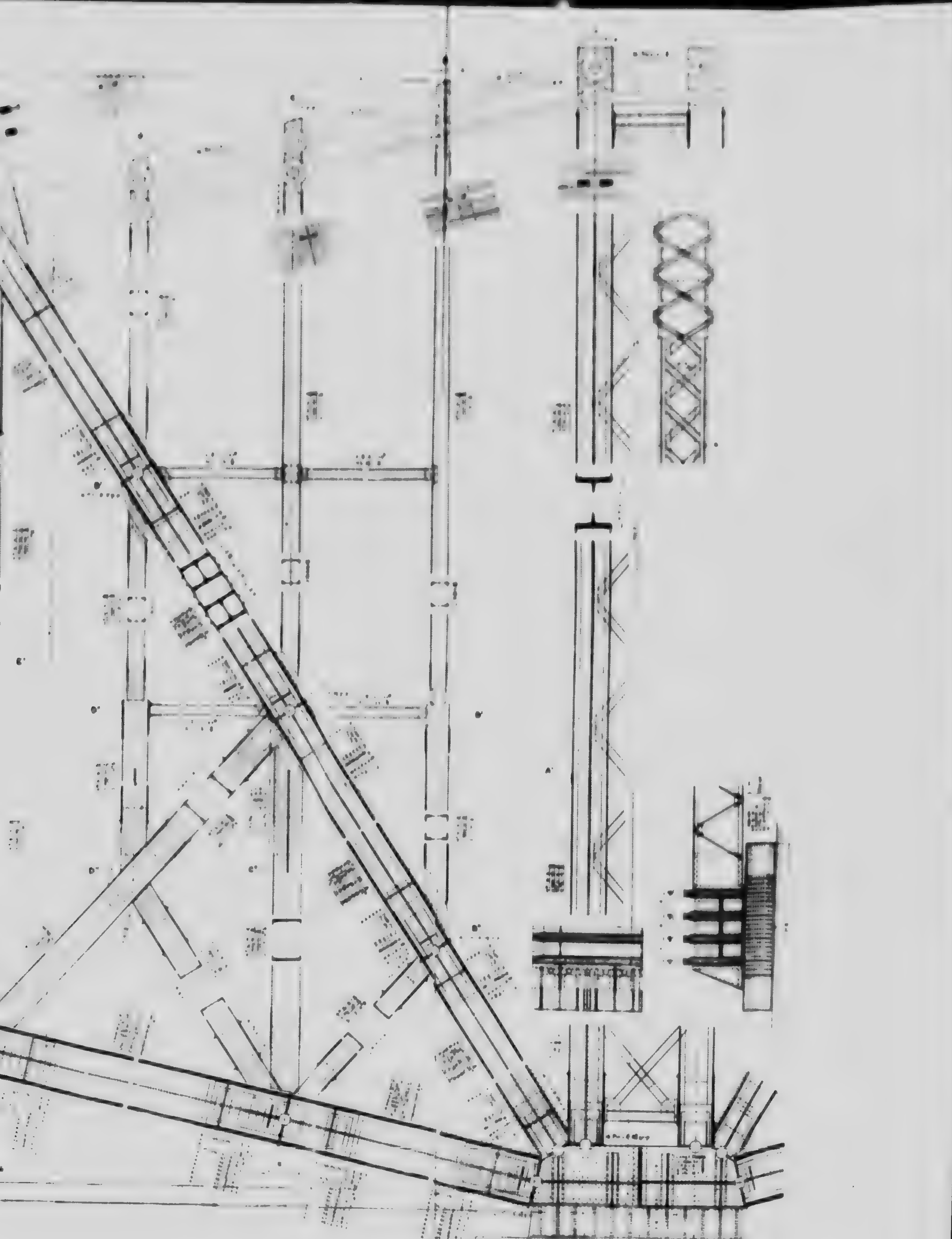
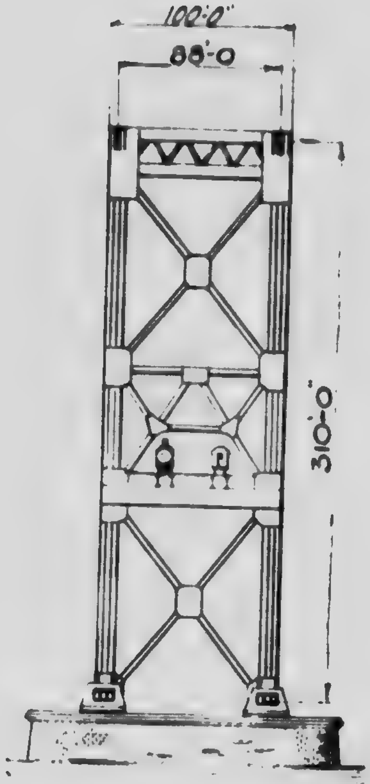
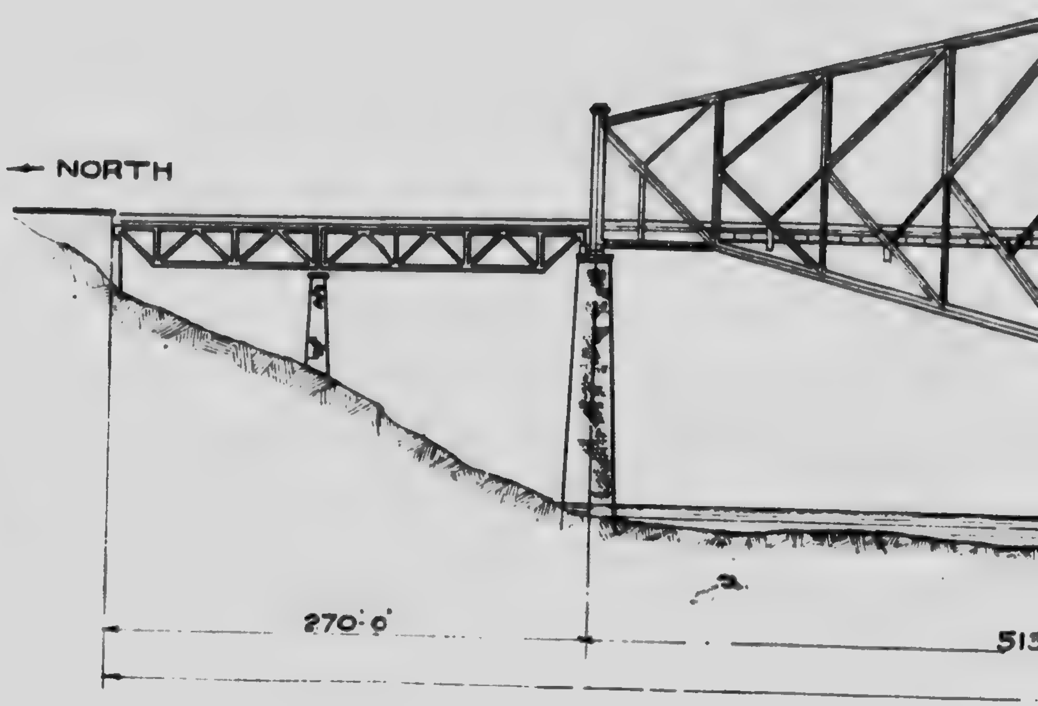
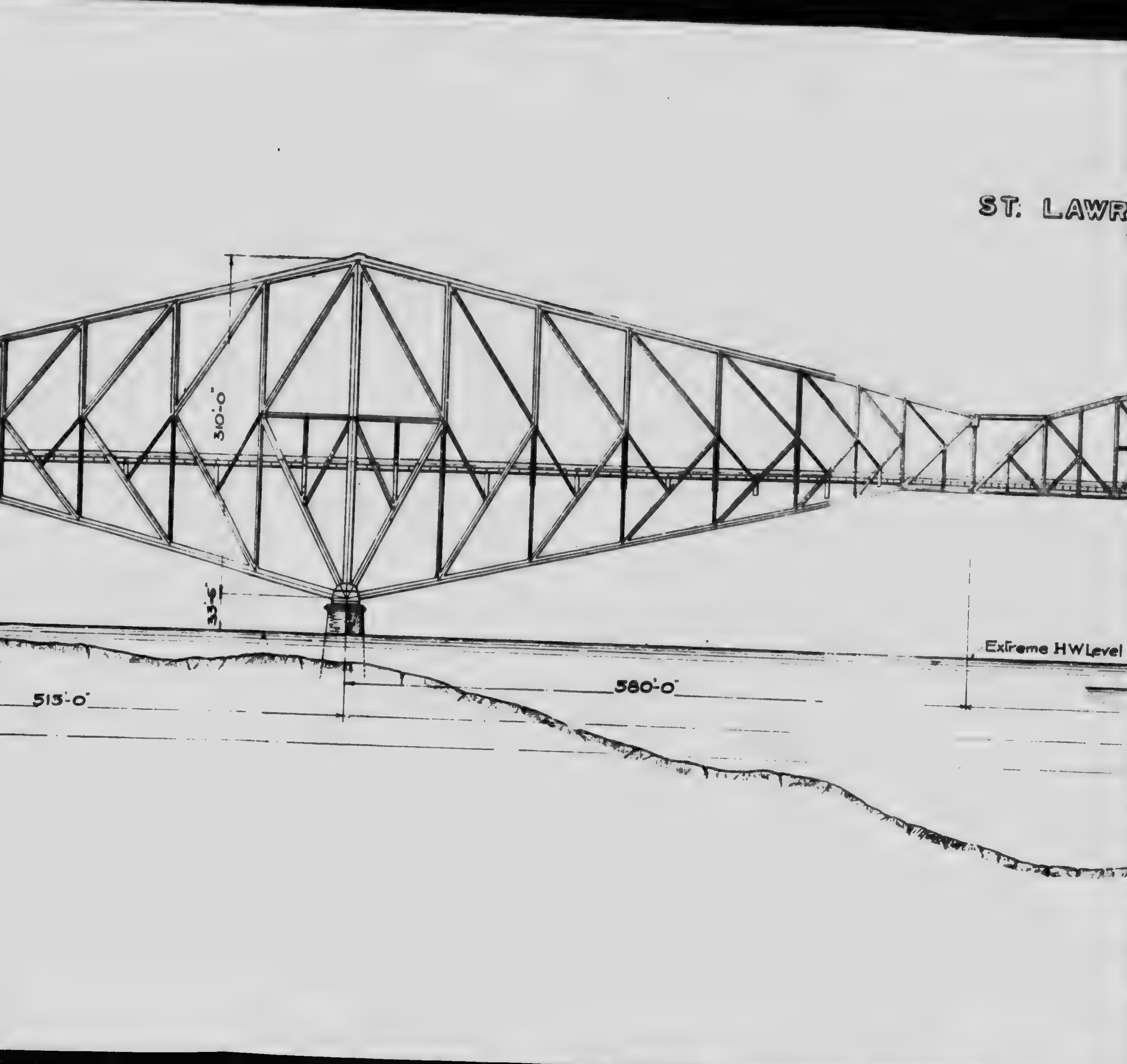


PLATE XVI

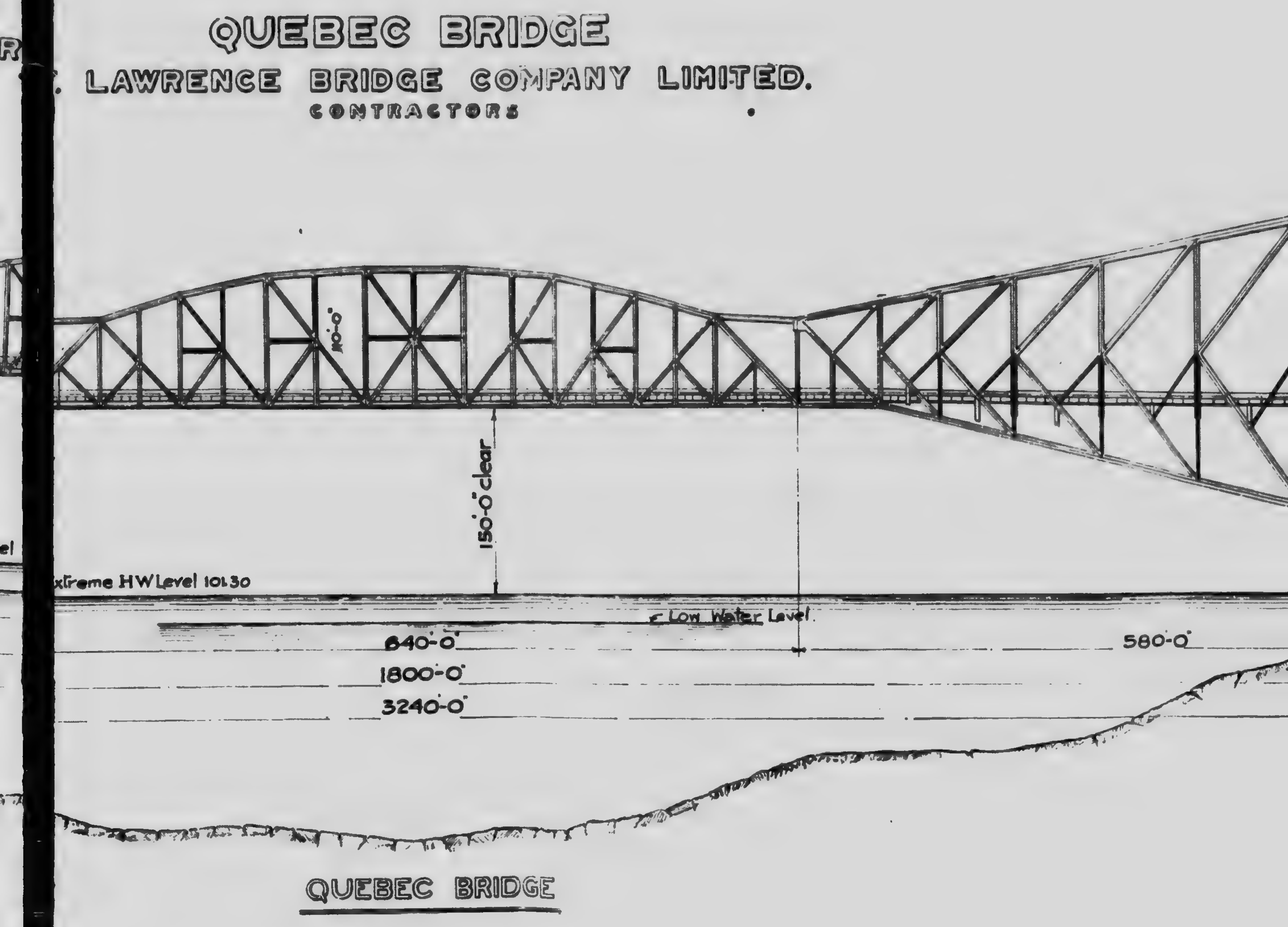


CROSS SECTION
at
MAIN PIERS



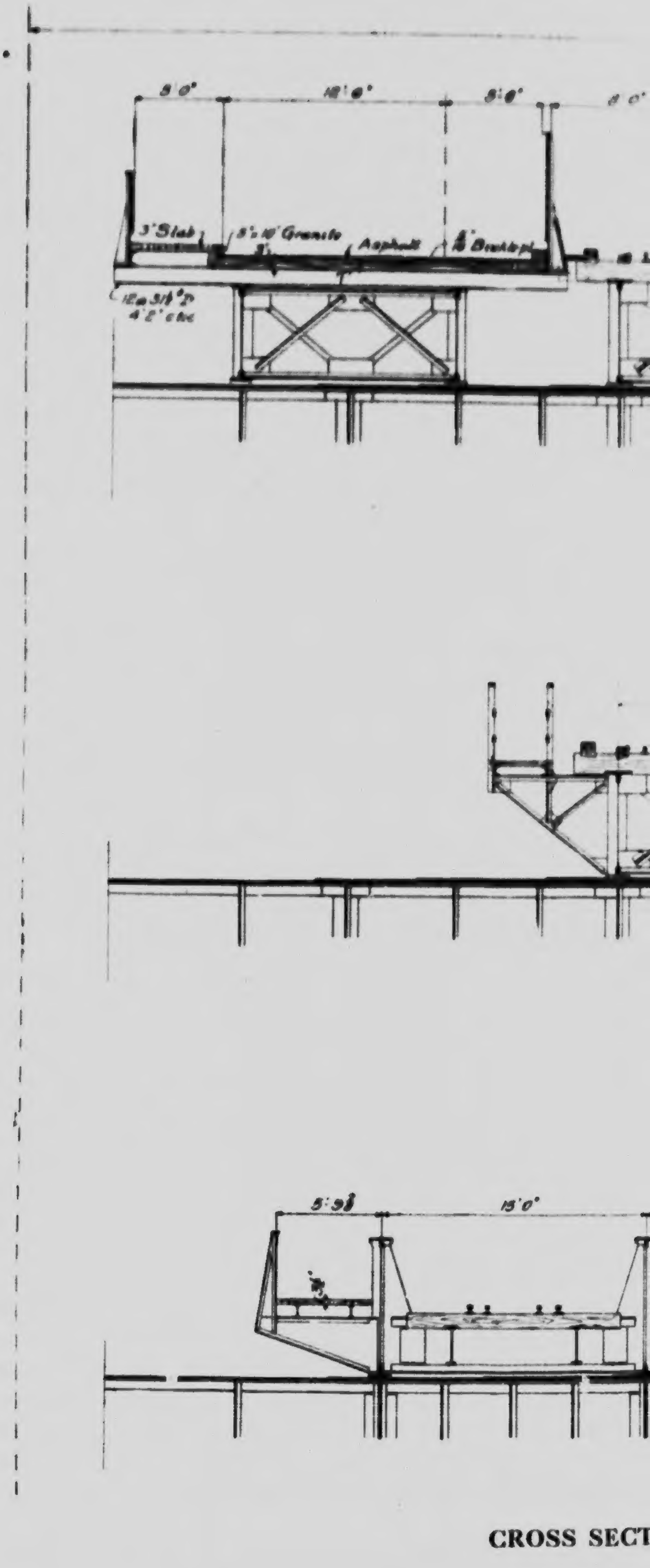


ST. LAWR
310'-0"
33'-6"
Extreme HW Level
513'-0"
580'-0"

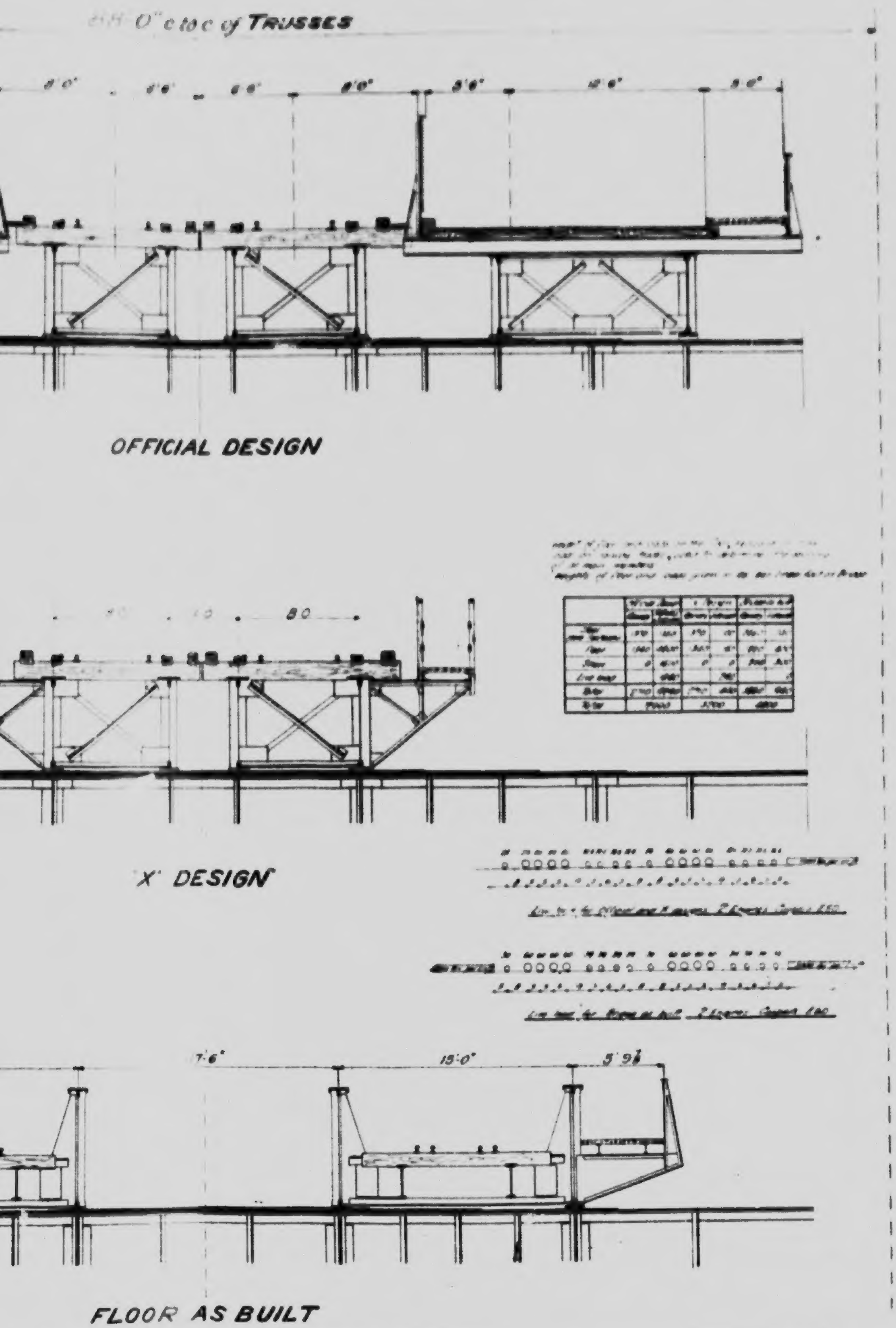


GENERAL ELEVATION OF BRIDGE AS BUILT

PLATE XVII



CROSS SECT



SS SECTION OF FLOOR, OFFICIAL DESIGN

PLATE XVIII